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नेपाल भूकम्प प्रतिरोधी भवन निर्माण ढाँचा (डिजाइन)

SEISMIC DESIGN OF BUILDINGS IN NEPAL

नेपाल सरकार

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PART 1 SCOPE AND DEFINITIONS

1 Title, Scope, Definitions and Notations

1.1 Title

Nepal National Building Code NBC 105: Seismic Design of Buildings in Nepal is the title of this document. The document is the outcome of the revision of the earlier version of NBC 105: 1994 Seismic Design of Buildings in Nepal.

1.2 Scope

This code covers the requirements for seismic analysis and design of various building structures to be constructed in the territory of the Federal Republic of Nepal. This code is applicable to all buildings, low to high rise buildings, in general. Requirements of the provisions of this standard shall be applicable to buildings made of reinforced concrete, structural steel, steel concrete composite, timber and masonry.

For Base-isolated buildings as well as for buildings equipped and treated with structural control can be designed in reference with specialist literatures.

Minimum design earthquake forces for buildings, structures or components thereof shall be determined in accordance with the provisions of this standard. Some definitions and symbols pertinent to the earthquake resistant design for buildings are presented in Sections 1.3 and 1.4. Section 1.5 presents the units adopted in this standard.

1.3 Definitions

Some terminologies related to earthquake resistant design of buildings used in this code are defined as follows:

BASE: The level at which the inertia forces developed in the building structure are accumulated before being transferred to the ground through the foundation. It is considered to be at the bottommost level of basement, or at the top of pile cap, or at the top of raft, or at the top of the footing.

BASE SHEAR: Total design lateral force or shear force due to earthquake at the base of a structure.

BRACED FRAME: A structural system that involves additional elements known as braces to a frame structure in order to increase its ability to resist lateral loads.

CAPACITY DESIGN: A design method in which critical elements of the structural systems are chosen and suitably designed and detailed for energy dissipation under severe deformation while all other structural elements are provided with sufficient strength so that the chosen means of energy dissipation can be maintained.

CENTER OF MASS (CM): The point in a floor through which the resultant of the mass passes.

CENTER OF STIFFNESS/RIGIDITY (CR): The point in a floor at which the resultants of the resisting forces in the two orthogonal directions intersect.

CRITICAL DAMPING: The damping beyond which the free vibration motion will not be oscillatory.

DAMPING: The effect of inherent energy dissipation mechanisms in a structure due to internal friction, inelasticity of materials, slipping, sliding etc. that results in reduction of oscillation amplitudes, expressed as a percentage of the critical damping for the structure.

DEAD LOAD: The weight of all permanent components of a building including walls, partitions, columns, beams, floors, roofs, finishes and fixed plant and fittings that are integral parts of the structure.

DESIGN ACCELERATION RESPONSE SPECTRUM: Average smoothened idealized plot of maximum pseudo-acceleration response to the design basis earthquake excitation applied at the base of a single degree of freedom system (representing the structure) as a function of the natural period and damping ratio of the structure.

DESIGN BASIS EARTHQUAKE: The earthquake ground motion considered for normal design, taken as two-thirds of the corresponding Maximum Considered Earthquake (MCE).

DIAPHRAGM: A horizontal or nearly horizontal system of structures acting to transmit lateral forces to the vertical resisting elements. The term "diaphragm" includes reinforced concrete floor slabs as well as horizontal bracing systems.

DUAL SYSTEM: A combination of a Moment Resisting Frame and Shear Walls or Braced Frames, where moment resisting frames are designed to resist independently at least 25% of the design base shear.

DUCTILITY: Capacity of a structure, or its members to undergo large inelastic deformations without significant loss of strength.

DUCTILITY FACTOR: The ratio of ultimate displacement demand to yield displacement demand.

ECCENTRICITY: The distance between the center of mass and center of stiffness.

EPICENTRE: The geographical point on the surface of earth vertically above the point of origin (focus) of the earthquake.

FLEXIBLE DIAPHRAGM: A floor or roof diaphragm, in which the maximum lateral deformation of the diaphragm is more than two times the average story drift of the associated story.

FLEXIBLE ELEMENT OR SYSTEM: An element or system whose deformation under lateral load is significantly larger than adjoining parts of the system.

IMPORTANCE FACTOR: A factor used to adjust the design seismic forces depending on the functional use of the structure.

INTENSITY OF EARTHQUAKE: It is a measure of the severity of ground shaking at a particular site due to an earthquake

INTERSTOREY DRIFT: Relative displacement of adjacent floors.

LIQUEFACTION: State in saturated cohesion less soil wherein the effective shear strength is reduced to negligible value due to pore water pressure generated by earthquake vibrations, when the pore water pressure approaches the total confining pressure. In this condition, the soil tends to behave like a liquid.

LIVE LOAD: The load assumed or known to result from the occupancy or use of a building and includes the loads on floors, loads on roofs other than wind, loads on

balustrades, and loads from movable goods, machinery, and plants that are not integral parts of the building.

MAGNITUDE OF EARTHQUAKE: A measure of the energy released at the source of an earthquake.

MAXIMUM CONSIDERED EARTHQUAKE (MCE): The most severe earthquake ground motion likely to occur at a location.

MODAL MASS: Part of the total seismic mass of the structure that is effective in a specific mode of vibration.

MODAL PARTICIPATION FACTOR: Amount by which a specific vibration mode contributes to the overall response of the structure.

MOMENT RESISTING FRAME: Assembly of beams and columns that resist internally produced and externally applied forces primarily by flexure and are specially detailed for ductility.

NATURAL PERIOD OF BUILDING: The time taken in seconds by the structure to complete one cycle of oscillation in its fundamental mode of response.

OVERSTRENGTH FACTOR: The ratio of the first significant yield strength of structure to the design base shear of the structure.

P-DELTA EFFECT: Structural actions induced as a consequence of the gravity loads being displaced horizontally due to horizontal actions.

PEAK GROUND ACCELERATION (PGA): Maximum acceleration of the ground in a particular direction of the ground motion.

RESPONSE SPECTRUM: Plot of the maximum response of a SDOF system versus its fundamental time period for a given level of damping (commonly taken as 5% of critical).

SHEAR WALL: A wall designed to resist lateral forces acting in its own plane.

SOFT STORY: Story in which the lateral stiffness is less than 70 per cent of the stiffness of the story above or less than 80 percent of the average lateral stiffness of the three stories above.

1.4 NOTATIONS

The following symbols and notation shall apply to the provisions of this section:

$\mathbf{a_p}$	Component Amplification Factor
$\mathbf{A_w}$	Area of Web
b	Maximum horizontal dimension of the building at the particular level measured perpendicular to the direction of loading
CQC	Complete Quadratic Combination
C(T)	Elastic Site Spectra for horizontal loading
$\mathbf{C_d(T)}$	Horizontal Design Spectrum
$\mathbf{C_d(T_1)}$	Horizontal Base Shear Coefficient
$\mathbf{C_h(T)}$	Spectral Shape Factor
$\mathbf{C_s(T)}$	Elastic Site Spectra for Serviceability Limit State
$\mathbf{C_v(T)}$	Elastic Site Spectra for Vertical Loading
$\mathbf{C_d(T_i)}$	Ordinate of the design spectrum for translational period Ti
DL	Design dead load
d*	Displacement of equivalent SDOF system
$\mathbf{d_{et}^*}$	Target displacement of a structure with period T*
$\mathbf{d_i}$	Horizontal displacement of the center of mass at level i, ignoring the effect of Torsion
$\mathbf{d_y^*}$	Displacement at yield of idealized SDOF system
E	Design earthquake load
ESM	Equivalent Static Method
$\mathbf{e_c}$	Computed distance between the center of mass and the center of rigidity

e_d	Design eccentricity of the seismic force at a particular level
E_c	Modulus of Elasticity of Concrete
E_x	Design earthquake load in Principal direction X
E_y	Design earthquake load in Principal direction Y
F^*	Force of equivalent SDOF system
F_i	Lateral force acting at level i
F_p	Design seismic force for parts and components
F_y^*	Yield force of idealized SDOF system
g	Acceleration due to gravity. To be taken as 9.81 m/s^2
H	Height from the base to the top of the main portion of the building or the eaves of the building (m)
h_i	Height of the level i from the base considered
h_p	Height of the component
i	level under consideration of the structure
I	Importance factor for the building
I_p	Component Importance Factor
I_g	Section moment of inertia calculated using the gross cross sectional area of concrete
LL	Design live load
m^*	Mass of equivalent SDOF system
MRSM	Modal Response Spectrum Method
n	Number of levels in a structure
PGA	Peak Ground Acceleration
RC	Reinforced Concrete

R_S Ductility Factor for Serviceability Limit State

R_μ Ductility Factor for Ultimate Limit State

SDOF Single degree of freedom

SRSS Square root of sum of squares

T* Time period of idealized equivalent SDOF system

T_1 Approximate Fundamental Period of Vibration

T_c Corner period corresponding to the end of constant spectral acceleration range

T_i Fundamental Translation Period of i^{th} mode of vibration

T Period of vibration of a linear single degree of freedom system

V Horizontal seismic base shear obtained from equivalent static method

V_R Combined base shear obtained from modal response spectrum method

W Seismic weight of the structure

W_i Seismic weight at level I;

W_p Component weight

w_j portion of gravity load at story level j

Z Seismic zoning factor

Ω_s Overstrength factor for serviceability limit state

Ω_u Overstrength factor for ultimate limit state

α Peak spectral acceleration normalized by PGA

μ_p Component ductility factor

ϕ_{ji} mode shape coefficient at floor j in mode i

1.5 UNITS

Unless otherwise noted, this code uses SI units of kilograms, metres, seconds, Pascals and Newton (kg, m, s, Pa, N).

2 General Principles

2.1 PERFORMANCE REQUIREMENTS AND VERIFICATION

Structures designed and built in seismic regions shall fulfil following fundamental requirements:

2.1.1 Life Safety:

The structure shall be designed and constructed to withstand the design seismic forces without local or global failure that, thus retaining its structural integrity, stability against overturning and a residual load bearing capacity after the earthquake. Further, it is also necessary to avoid damage to non-structural systems which are essential for safe evacuation from the structure.

The design seismic force is expressed in terms of 475 years return period (reference return period) and the importance factor.

2.1.2 Damage Limitation:

The structure shall be designed and constructed to withstand a seismic force having a larger probability of occurrence than the design seismic forces, without the occurrence of damage and the associated limitations of use of the structure. The critical facilities need to be operational state or be in a state which can be returned to fully operational state shortly after the earthquake (within minutes to hours).

The design seismic force associated with damage limitation is expressed in terms of a fraction of life safety level seismic force.

2.1.3 VERIFICATION:

For the verification of the performance requirements of clause, following limit states shall be checked:

Ultimate Limit State (ULS);

Serviceability Limit State (SLS).

2.1.3.1 Ultimate Limit State Verification:

Ultimate limit states are associated with collapse or with other forms of structural failure which might endanger the safety of people. Design for

ultimate limit state represents a procedure that ensures the probability of collapse of a structure is at an acceptable level.

The ultimate limit state performance requirements are met when the structure satisfies the following:

1. The structural system has the required resistance and energy dissipation capacity;

2. The structure as a whole shall be checked to ensure that it is stable under the design seismic forces. Both overturning and sliding stability shall be taken into account;

3. The structural system shall continue to perform its load-bearing function;

4. Both the foundation elements and the foundation soil are able to resist the forces resulting from the response of the superstructure without substantial permanent deformations;

5. Non-structural systems which are essential for safe evacuation from the structure shall continue to function;

6. The nonstructural elements do not present risks to people and does not have a detrimental effect on the response of the structural elements.

2.1.3.2 Serviceability Limit State Verification:

Damage limitation states are associated with damage beyond which specified service requirements are no longer met. It represents a level of force within the structure below which there is a high degree of assurance that the structure can continue to be used as originally intended without repair.

The serviceability limit state performance requirements are met when the structure satisfies the following:

1. The structural system shall not experience deformations that result in structural or non-structural damage that can prevent the structure from performing its intended original function.

2. In structures important for civil protection, the structure shall have sufficient resistance and stiffness to remain operational so that it can perform its function of the vital services in the event of an earthquake.

2.2 GENERAL GUIDELINES FOR ARRANGEMENT OF BUILDING STRUCTURAL SYSTEMS

2.2.1 Structural simplicity

Structural simplicity is characterized by the existence of clear and direct paths for the transmission of the seismic forces. Modeling, analysis, dimensioning, detailing and construction of simple structures are subject to much less uncertainty and thus the prediction of their seismic behavior is much more reliable.

2.2.2 Uniformity, symmetry and redundancy

A. Uniformity in plan is characterized by an even distribution of the structural elements which allows direct transmission of the inertia forces induced in the distributed masses of the building. If necessary, uniformity may be realized by subdividing the entire building by seismic joints into dynamically independent units, provided that these joints are designed against pounding between the individual units. Uniformity of the structure along the height of the building is also essential, as it tends to eliminate the occurrence of sensitive zones where high stress or ductility demands might concentrate.

B. A similarity between the distribution of masses and the distribution of resistance and stiffness eliminates large eccentricities between mass and stiffness.

C. If the building configuration is symmetrical or quasi-symmetrical, a symmetrical layout of structural elements, which should be well-distributed in-plan, is appropriate for the achievement of uniformity.

D. The use of evenly distributed structural elements increases redundancy allows a more favorable redistribution of member forces and spreads the energy dissipation widely across the entire structure.

2.2.3 Adequate resistance and stiffness

A. Horizontal seismic motion is a bi-directional phenomenon and thus the building structure shall be able to resist horizontal actions in any direction. In this respect, structural elements should be arranged in an orthogonal in-plan structural pattern, ensuring adequate resistance and stiffness characteristics in both main directions.

B. In addition to lateral resistance and stiffness, building structures should possess adequate torsional resistance and stiffness in order to limit the development of torsional motions which tend to stress different structural elements in a non-uniform way. In this respect, arrangements in which the main elements resisting the seismic action are distributed close to the periphery of the building present clear advantages.

2.2.4 Diaphragm action

A. In buildings, floors (including the roof) play a very important role in the overall seismic behavior of the structure. They act as horizontal diaphragms that collect and transmit the inertia forces to the vertical structural systems and ensure that those systems act together in resisting the horizontal seismic action. The action of floors as diaphragms is especially relevant in cases of complex and non-uniform layouts of the vertical structural systems, or where systems with different horizontal deformability characteristics are used together (e.g. in dual or mixed systems).

B. Floor systems and the roof should be provided with in-plane stiffness and resistance and with effective connection to the vertical structural systems. Particular care should be taken in cases of non-compact or very elongated in-plan shapes and in cases of large floor openings, especially if the latter are located in the vicinity of the main vertical structural elements, thus hindering such effective connection between the vertical and horizontal elements.

C. Diaphragms should have sufficient in-plane stiffness for the distribution of horizontal inertia forces to the vertical structural systems in accordance with the assumptions of the analysis, particularly when there are significant

changes in stiffness or offsets of vertical elements above and below the diaphragm.

D. The diaphragm may be taken as being rigid, if, when it is modeled with its actual in-plane flexibility, its horizontal displacements nowhere exceed those resulting from the rigid diaphragm assumption by more than 10% of the corresponding absolute horizontal displacements under seismic forces.

E. The connection between a diaphragm and the surrounding vertical structural system in each direction should be able to resist 1.2 times the design inertial force induced at that level.

2.2.5 Adequate foundation

A. With regard to the seismic action, the design and construction of the foundations and their connection to the superstructure shall ensure that the whole building is subjected to a uniform seismic excitation.

B. For buildings with individual foundation elements (footings or piles), the use of a foundation slab or tie-beams between these elements in both main directions is recommended.

2.3 RESPONSE TO EARTHQUAKE GROUND MOTION

2.3.1 Ground Motion

The earthquake ground motion intensity at a site depends on the magnitude of the earthquake, the earthquake source to the site distance and the medium of the seismic waves travel path. The random ground motion due to an earthquake can be generally resolved into three orthogonal directions, the dominant components normally being in horizontal directions.

2.3.2 Response of Structure

The response of a building to a seismic ground motion depends on the structural system as well as the principal parameters of the ground motion. This standard considers the seismic design forces for buildings located on soils, which do not settle, liquefy or slide during earthquakes.

2.3.3 Soil-structure Interaction

The soil-structure interaction, representing the effects of the flexibility of supporting soil foundation system on the response of the building, may not be considered in the earthquake analysis of buildings supported on hard soils or rocks.

Where the soil has been identified as soft and prone to liquefaction and/or lateral spreading, design of foundation shall account for the potential soil movement. Deep (i.e. pile) foundations shall be normally preferred in such sites. Wherever needed, advice should be sought from a geotechnical engineering expert.

2.4 CAPACITY DESIGN

Capacity design shall be applied to ductile structures and other structures where required by the appropriate material standards.

2.4.1 Potential Plastic Zones

Ductile failure modes for the proposed structure shall be identified for each potential direction of seismic actions. The location of all potential plastic zones shall be identified and proportioned so that the design strength exceeds the design actions at these locations.

2.4.2 Level of Detailing

The level of detailing required to sustain the material strain levels in the critical potential plastic zones when subjected to displacements defined in section 5.6 shall be determined.

2.4.3 Overstrength Actions

The maximum likely strength actions in potential plastic zones shall be derived considering possible Overstrength factors.

2.5 BASIC ASSUMPTIONS

It is generally asssumed in the seismic design of buildings that:

(a) A severe earthquake does not occur simultaneously with a strong wind, or a high flood. Hence, the effect of only the earthquake with the dead

and live loads resulting into the largest effect in the building is taken into consideration.

(b) The modulus of elasticity of the materials of construction, when required in the design, will be taken as that for static analysis, unless more definite dynamic values are available.

PART 2 STRUCTURAL ANALYSIS AND DESIGN

3 Scope of Analysis

3.1 STRUCTURAL ANALYSIS METHODS

The structural analysis for design seismic actions shall be carried out using any one of the following methods:

a) Equivalent Static Method

b) Linear Dynamic Analysis Methods

   i. Modal Response Spectrum Method

   ii. Elastic Time History Analysis

c) Non-linear Methods

   i. Non-linear Static Analysis

   ii. Non-linear Time History Analysis

3.2 APPLICABILITY OF ANALYSIS METHODS

3.2.1 Equivalent Static Method (ESM)

The Equivalent Static Method may be used for all serviceability limit state (SLS) calculations regardless of the building characteristics.

For ultimate limit state (ULS), the Equivalent Static Method may be used when at least one of the following criteria is satisfied:

i. The height of the structure is less than or equal to 15 m.

ii. The natural time period of the structure is less than 0.5 secs.

iii. The structure is not categorized as irregular as per **5.5** and the height is less than 40 m.

3.2.2 Modal Response Spectrum Method (MRSM)

The Modal Response Spectrum Method may be used for all types of structures and the structures where Equivalent Static Method is not applicable. A three dimensional analysis shall be performed for torsionally sensitive structures.

3.2.3 Elastic Time History Analysis

The elastic time history analysis may be used for all types of structures to verify that the specific response parameters are within the limits of acceptability assumed during design. A three dimensional analysis shall be performed for torsionally sensitive structures.

3.2.4 Non-linear Methods

As an alternative to linear analysis methods, nonlinear analysis methods may be used for structural analysis. Following two analysis methods are prescribed in the present standard:

i. Non-linear Static Analysis
ii. Non-linear Dynamic Analysis

These methods are used basically to verify the performance of existing or retrofitted structures. These methods can also be used to verify that the specific response parameters are within the limits of acceptability assumed during design.

3.3 APPLICATION OF SEISMIC FORCES

Seismic forces in a structure shall be considered in each of the two principal directions of the structure. For structures which have the lateral force resisting elements in two orthogonal directions, the design forces shall be considered in one orthogonal direction only at a time.

3.4 EFFECTIVE STIFFNESS OF CRACKED SECTIONS

A rational analysis shall be performed in arriving at the elastic flexural and shear stiffness properties of cracked concrete and masonry elements. In absence of such analysis, the effective stiffness of cracked sections shall be taken from Table 3-1.

Table 3-1 Effective stiffness of different components

S No.	Component	Flexural Stiffness	Shear Stiffness
1	Beam	$0.35 E_c I_g$	$0.40 E_c A_w$
2	Columns	$0.70 E_c I_g$	$0.40 E_c A_w$
3	Wall—cracked	$0.50 E_c I_g$	$0.40 E_c A_w$
4	Wall—uncracked	$0.80 E_c I_g$	$0.40 E_c A_w$

For steel structures, the gross stiffness values shall be used.

3.5 DESIGN METHODS

Design for earthquake actions shall be in accordance with Limit State Method (LSM).

3.6 LOAD COMBINATIONS FOR LIMIT STATE METHOD

3.6.1 Load Combinations for Parallel Systems

Where seismic load effect is combined with other load effects, the following load combination shall be adopted.

1.2DL + 1.5LL

$DL + \lambda LL \pm E$ .....(3.6.1)

Where, λ = 0.6 for storage facilities

= 0.3 for other usage

3.6.2 Load Combinations for Non- Parallel Systems

When lateral load resisting elements are not oriented along mutually orthogonal horizontal directions, structure shall be designed for the simultaneous effects due to full design earthquake load in one direction plus 30 percent of design earthquake load along the other horizontal direction. In this case, the following load combination shall be adopted.

1.2DL + 1.5LL

$DL + \lambda LL \pm (E_x \pm 0.3E_y)$

$DL + \lambda LL \pm (0.3E_x \pm E_y)$ …….. (3.6.2)

Where, λ = 0.6 for storage facilities

= 0.3 for other usage

4 Seismic Hazard

4.1 ELASTIC SITE SPECTRA FOR HORIZONTAL LOADING

4.1.1 Elastic site spectra

The Elastic site spectra for horizontal loading shall be as given by equation 4.1(1).

$C(T) = C_h(T)\ Z\ I$ .....4.1(1)

Where,

$C_h(T)$ = Spectral Shape factor as per 4.1.2

Z = Seismic Zoning factor as per 4.1.4

I = Importance factor as per 4.1.5

4.1.2 Spectral Shape Factor, $C_h(T)$

The Spectral Shape Factor, $C_h(T)$ for the relevant soil type shall be obtained either from Figure 4-1 and Figure 4-2, or calculated by equation 4.1(2) using the parameters specified in Table 4.1. The spectral shape factor functions given in Figure 4-1(a) shall be used for Equivalent Static Method and those in Figure 4-1(b) shall be used for Modal Response Spectrum Method and Nonlinear Time History Analysis.

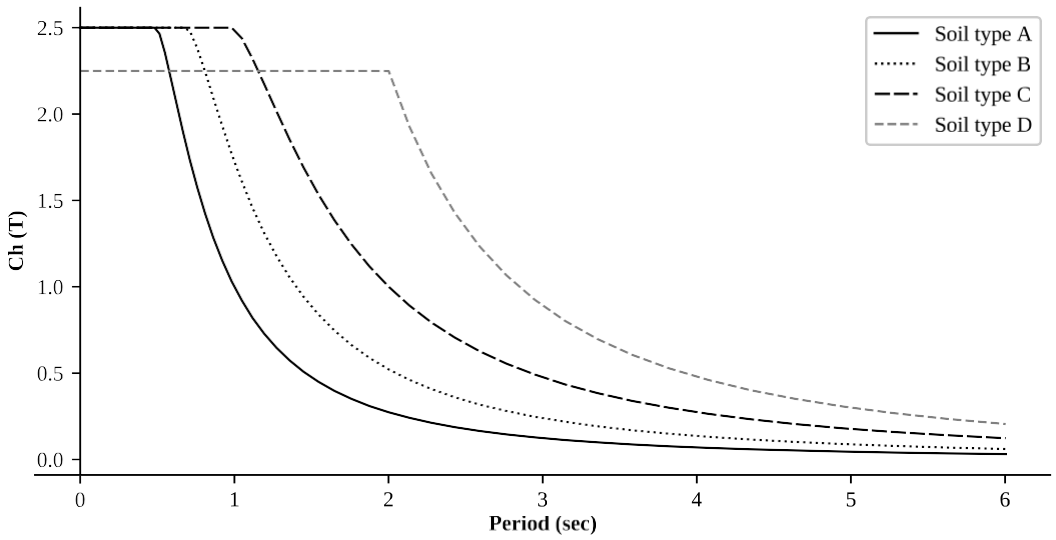


Figure 4-1Spectral Shape Factor, $C_h(T)$ for Equivalent Static Method

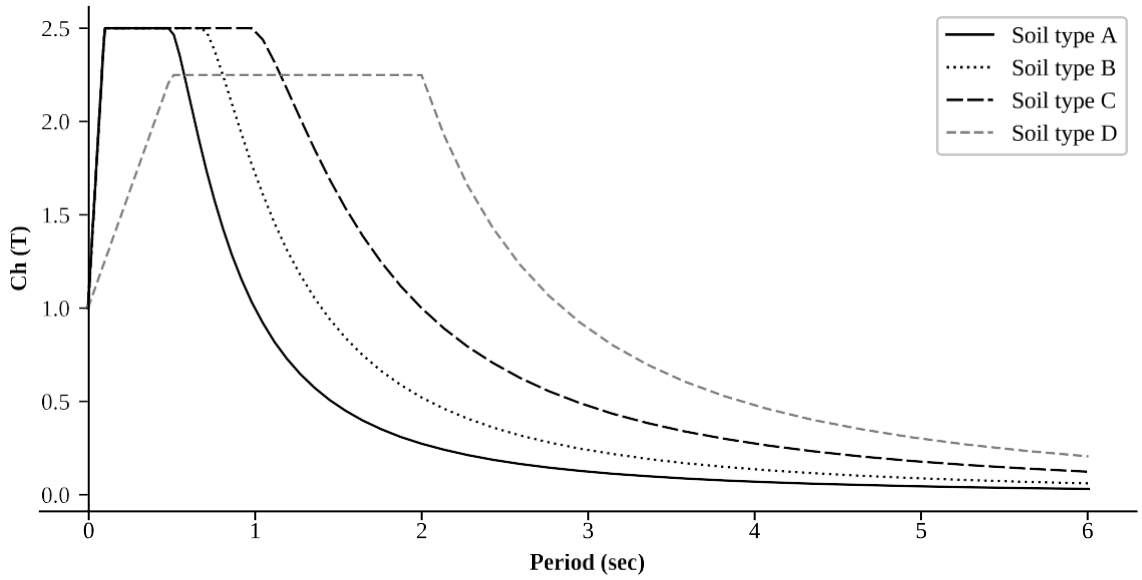


Figure 4-2 Spectral Shape Factor, $C_h(T)$ for Modal Response Spectrum Method, Nonlinear Time History Analysis , Vertical Loading and Parts and Components

$$C_h(T) = \begin{cases} 1 + (\alpha - 1) \times \frac{T}{T_a} \; if \; T < T_a \\ \alpha \; if \; T_a \leq T \leq T_c \\ \alpha \left[K + (1 - K) \left(\frac{T_c}{T} \right)^2 \right] \left(\frac{T_c}{T} \right)^2 \; if \; T_c \leq T \leq 6 \end{cases} \quad \ldots 4.1(2)$$

Where,

α - peak spectral acceleration normalized by PGA

T_a and T_c - the lower and upper periods of the flat part of the spectrum

K – Coefficient that controls the descending branch of the spectrum

Table 4-1 *T_a, T_c, α & K*

Parameters\Soil Type	Soil Type A	Soil Type B	Soil Type C	Soil Type D
T_a	0.1[1]	0.1[1]	0.1[1]	0.5[1]
T_c	0.5	0.7	1.0	2.0
α	2.5	2.5	2.5	2.25
K	1.8	1.8	1.8	0.8

[1] The value of T_a shall correspond to zero for Equivalent Static Method.

4.1.3 Site Subsoil Category

The site subsoil class shall be determined as one of the Soil Types from 4.1.3.1 to 4.1.3.4:

4.1.3.1 Soil Type A - Stiff or Hard Soil Sites

Sites with bedrock, including weathered rock with an unconfined compressive strength greater than 500 kPa, overlain by less than 20 m of:

I. Very stiff cohesive material with an unconfined compressive strength greater than 100 kPa, or
II. Very dense cohesionless material with N > 30, where N is the standard penetration test (SPT) value.

Such sites will have typically a low amplitude natural period less than 0.2 s.

4.1.3.2 Soil Type B - Medium Soil Sites

Sites not described as either **Soil Type A, C** or **D**.

Sites where the depth of soil does not exceed those stipulated in **Table 4-2** and **Table 4-3**.

Such sites will have typically a low amplitude natural period less than 0.6 s.

4.1.3.3 Soil Type C - Soft Soil Sites

Sites where the depth of soil of a particular type exceeds those stipulated in **Table 4-2** and **Table 4-3**.
Such sites will have typically a low amplitude natural period greater than 0.6 s.

***Table 4-2**: Cohesive Soils*

Cohesive Soil Classification	Representative undrained shear strength (kPa)	Minimum Depth of Soil (m)
Soft	12.5 - 25	20
Firm	25 - 50	25
Stiff	50 - 100	40
Very Stiff	100 - 200	60

Table 4-3: Cohesionless Soils

Cohesionless Soils Classification	Representative SPT values (N)	Minimum Depth of Soil (m)
Loose	4 - 10	40
Medium Dense	10 - 30	45
Dense	30 - 50	55
Very Dense	> 50	60
Gravels	> 30	100

4.1.3.4 Soil Type D - Very Soft Soil Sites

Soil site which comprises of:

I. More than 10 m depth of cohesive soil with undrained shear strength less than 12.5 kPa
II. More than 10 m depth of cohesionless soil with SPT N-values less than 4

Such sites will have typically a low amplitude natural period greater than 1.0 sec.

Sites located inside Kathmandu valley shall be obtained from Table 4-4.

Table 4-4: Type D Soil sites

S. No.	Municipalities
1	Kathmandu
2	Lalitpur
3	Bhaktapur
4	Madhyapur Thimi
5	Kageshori Manohara
6	Tokha

4.1.4 Seismic Zoning Factor (Z)

The country is subdivided into different seismic zones based on the local seismic hazard. The seismic hazard within each zone is assumed to be constant. The Seismic Zoning Factor (Z) represents the peak ground acceleration (PGA) for 475 year return period. The value of Z can be obtained from **Table 4-5** for selected municipalities, cities and for the rest can be obtained from **Figure 4-4** (approximate interpolation between the contour lines is permitted).

***Table 4-5:** Seismic Zoning factors for selected cities and municipalities*

Cities/Municipalities	PGA	Cities/Municipalities	PGA
Baglung	0.3	Janakpur	0.3
Beni	0.3	Jomsom	0.25
Besishar	0.3	Jumla	0.3
Bharatpur	0.4	Kalaiya	0.3
Bhimdatta	0.3	Kamalamai	0.4
Bhimeshwar	0.3	Kapilbastu	0.3
Bhojpur	0.35	Kathmandu	0.35
Bidur	0.3	Khalanga	0.3
Biratnagar	0.3	Khandbari	0.3
Birendranagar	0.35	Kusma	0.3
Birgunj	0.3	Lahan	0.3
Butwal	0.3	Libang	0.35
Chainpur	0.3	Malangwa	0.3
Chame	0.25	Mangalsen	0.35
Chautara	0.3	Manma	0.3
Dadheldhura	0.35	Manthali	0.3
Dailekh	0.35	Martadi	0.3
Damak	0.3	Musikot	0.3

Damauli	0.35	Myanglung	0.35
Darchula	0.3	Nepalgunj	0.4
Dasharathchand	0.35	Okhaldhunga	0.35
Dhading	0.3	Phidim	0.35
Dhangadhi	0.4	Pokhara	0.3
Dhankuta	0.4	Pyuthan	0.35
Dharan	0.3	Rajbiraj	0.3
Dhulikhel	0.35	Ramgram	0.4
Dhunche	0.3	Salleri	0.3
Diktel	0.35	Salyan	0.35
Dipayal	0.35	Sandhikharka	0.35
Dunai	0.25	Simikot	0.25
Gamgadhi	0.25	Tamghas	0.35
Gaur	0.3	Tansen	0.35
Gorkha	0.3	Taplejung	0.3
Gulariya	0.4	Triyuga	0.4
Hetauda	0.4	Tulsipur	0.4
Ilam	0.4	Waling	0.35
Jaleshwor	0.3		

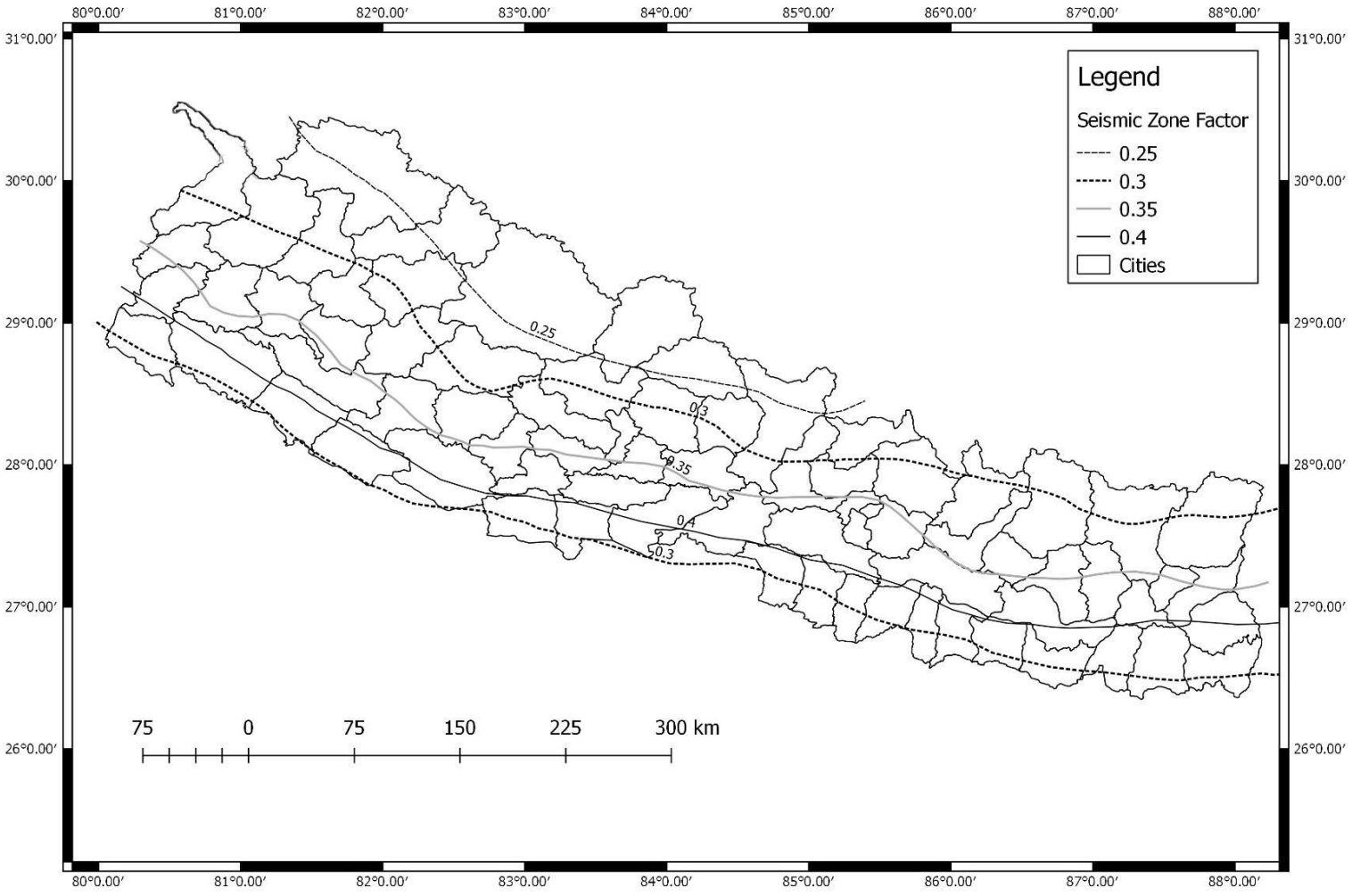


***Figure 4-3**: Seismic Zoning Map of Nepal*

4.1.5 Importance Classes and Importance Factor (I)

Structures are categorized into three Importance classes depending on the consequences of their loss of function. The importance classes are characterized by an important factor, I.

The importance classes and Factors are tabulated in **Table 4-6**.

Table 4-6: Importance Class and Factors

Importance Class	Structure	I
I	Ordinary Structures (those not falling in classes II and III)	1.0
II [2]	Schools, colleges, cinemas, assembly buildings such as shopping malls, convention halls, temples, monumental structures, Police stations, Emergency vehicle shelters/garages, Food storage structures, Emergency relief stores, Water works and water towers, Radio and television facilities, Telephone exchanges and transmission facilities, Offices and residential quarters for senior personnel required for rescue and relief operations and any other buildings designed to accommodate more than 500 persons.	1.25
III	Hospitals, fire stations, police headquarters, power stations (including standby power-generating equipment for essential facilities), distribution facilities for gas or petroleum products, structures for support or containment of dangerous substances (such as acids, toxic substances, petroleum products)	1.5

[2] Importance factor of 1.5 shall be applied if the facilities listed in Importance Class II are to be used as a shelter in case of a disaster.

4.2 ELASTIC SITE SPECTRA FOR SERVICEABILITY LIMIT STATE

The elastic site spectra for Serviceability Limit State shall be given by:

C_s (T) = 0.20 C (T) .....4.2(1)

Where C (T) = elastic site spectra for horizontal loading determined from clause 4.1.1.

4.3 ELASTIC SITE SPECTRA FOR VERTICAL LOADING

The elastic site spectra for vertical loading $C_v(T_v)$ shall be given by:

$C_v(T_v) = 2/3\ Z$.....4.3(1)

Clauses for consideration of vertical acceleration are;

a) For horizontal or nearly horizontal structural members spanning 20m or more;

b) For horizontal or nearly horizontal cantilever components longer than 5m;

c) For horizontal or nearly horizontal pre-stressed components;

d) For beams supporting Columns;

e) In Base-Isolated Structures.

5 Dynamic Characteristics of Structures

5.1 PERIODS OF VIBRATION

The periods of vibration, T_i, shall be established from properly substantiated data, or computation, or both. The fundamental translation period shall be determined using following methods:

1. Rayleigh Method
2. Empirical Equations

The fundamental translation period of a building shall be estimated using the appropriate empirical equations listed in section 5.1.2. The approximate time period calculated in section 5.1.2 shall be modified as per section 5.1.3. The time period so modified shall be compared with the translation period computed from section 5.1.1 and the lesser value of the two shall be adopted for determining the design action.

5.1.1 Rayleigh Method

The fundamental translation period in the direction under consideration, T_1, shall be calculated as:

$$T_1 = 2\pi\sqrt{\frac{\sum_{i=1}^{n}(W_i d_i^2)}{g\sum_{i=1}^{n}(F_i d_i)}} \quad \text{.....5.1(1)}$$

Where

d_i = elastic horizontal displacement of center of mass at level i, ignoring the effects of torsion.

F_i = lateral force acting at level i

g = acceleration due to gravity

i = level under consideration

n = number of levels in the structure

W_i = seismic weight at level i

5.1.2 Empirical Equations

The approximate fundamental period of vibration, T_1, in seconds is determined from following empirical equation:

$$T_1 = k_t H^{3/4} \quad \text{..... 5.1(2)}$$

Where, k_t

= 0.075 for Moment resisting concrete frame

= 0.085 for Moment resisting structural steel frame

= 0.075 for Eccentrically braced structural steel frame

= 0.05 for all other structural systems

Where,

H = Height of the building from foundation or from top of a rigid basement.

5.1.3 Amplification of Approximate Period

The approximate fundamental time period calculated using empirical equation in section 5 1.2 shall be increased by a factor of 1.25.

5.2 SEISMIC WEIGHT

The seismic weight at each level, W_i, shall be taken as the sum of the dead loads and the factored seismic live loads between the mid-heights of adjacent stories.

The seismic live load shall be determined as given in **Table 5-1**.

***Table 5-1**: Live Load Categories and Factors*

Live Load Category	Factor (λ)
Storage	0.60
For Other Purpose	0.30
Roof	Nil

5.3 DUCTILITY FACTOR

5.3.1 Ultimate limit state

The Ductility Factor (R_μ) shall be chosen to be consistent with the structural system and the structural member/connection detailing. The values of R_Ω for various types of structures are tabulated in **Table 5-2**.

5.3.2 Serviceability limit state

The Ductility Factor (R_s) for serviceability limit state shall be taken as 1.

5.4 OVERSTRENGTH FACTOR

5.4.1 Ultimate limit state

The Over-strength factor (Ω_u) for ultimate limit state, which accounts for the extra reserve strength that is inherently present in structures, shall be adopted from **Table 5-2** for appropriate structural system.

5.4.2 Serviceability limit state

The Over-strength factor (Ω_s) for serviceability limit state shall also be adopted from **Table 5-2** for appropriate structural system.

***Table 5-2**: Ductility and Overstrength Factors*

S. No.	Structural System	R_μ	Ω_u	Ω_s
	Moment Resisting Frame Systems			
1	Steel Moment Resisting Frame	4	1.5	1.25
2	Reinforced Concrete Moment Resisting Frame	4	1.5	1.25
3	Steel + RC Composite Moment Resisting Frame	4	1.5	1.25
	Braced Frame Systems			
4	Steel Eccentrically Braced Frame	4	1.5	1.25
5	Steel + RC Composite Eccentrically Braced Frame	4	1.5	1.25
6	Steel Concentric Braced Frame	3	1.3	1.15
7	Steel + RC Composite Concentric Braced Frame	3	1.3	1.15
8	Steel Buckling Restraint Braces	4	1.5	1.25
	Structural Wall Systems			
9	RC Shear wall	3	1.3	1.15
10	Steel + RC Composite Shear Wall	3	1.3	1.15
11	Reinforced Masonry Shear wall	2.5	1.2	1.1
12	Confined Masonry wall	2.5	1.2	1.1
13	Unreinforced Masonry wall buildings with horizontal bands and vertical reinforcement bars at critical location	2.0	1.2	1.1
	Dual Systems			
14	Steel Eccentrically Braced Frame	4	1.5	1.25
15	Steel + RC Composite Eccentrically Braced Frame	4	1.5	1.25
16	Steel Concentric Braced Frame	3.5	1.4	1.2
17	Steel + RC Composite Concentric Braced Frame	3.5	1.4	1.2
18	Steel Buckling Restraint Braces	4	1.5	1.25
19	RC Shear wall	3.5	1.4	1.2
20	Steel + RC Composite Shear Wall	3.5	1.4	1.2

21	Reinforced Masonry Shear wall	2.5	1.2	1.1

5.5 STRUCTURAL IRREGULARITY

Structures with simple and regular configurations suffer much less damage during a large earthquake. Irregular structures on the other hand suffer heavy damage during a large earthquake. Therefore, efforts shall be made to make the structure as regular as possible. Any structure is considered irregular if any of the clauses. 5.5.1 to 5.5.2 are applicable.

5.5.1 Vertical Irregularity

5.5.1.1 Weak Story

A story is considered as weak story if the strength of the lateral force resisting system in that story is less than 80% of the strength of the story above.

5.5.1.2 Soft Story

A soft story is the one whose stiffness of the lateral-force-resisting system is less than 70% of the lateral-force-resisting system stiffness in an adjacent story above or below, or less than 80% of the average lateral-force-resisting system stiffness of the three stories above or below.

5.5.1.3 Vertical Geometric Irregularity

Vertical geometric irregularity shall be considered to exist if the horizontal dimension of lateral force resisting system in any story is more than 130% of that in an adjacent story.

5.5.1.4 In-Plane Discontinuity in Vertical Lateral Force Resisting Element Irregularity

It shall be considered to exist where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on supporting structural elements (Figure 5-1).

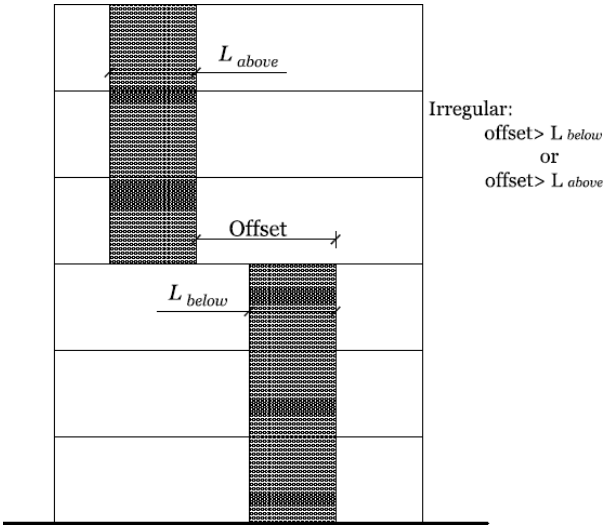


*Figure 5-1: **In-Plane Discontinuity***

5.5.1.5 Mass Irregularity

A difference of more than 50% between the effective masses of two consecutive stories is considered as mass irregularity. Light roofs, penthouse, and mezzanine floors need not be considered.

5.5.2 Plan Irregularity

5.5.2.1 Torsion Irregularity

Torsion irregularity is considered to exist where the maximum horizontal displacement of any floor in the direction of the lateral force (applied at the center of mass) at one end of the story is more than 1.5 times its minimum horizontal displacement at the far end of the same story in that direction (Figure 5-2).

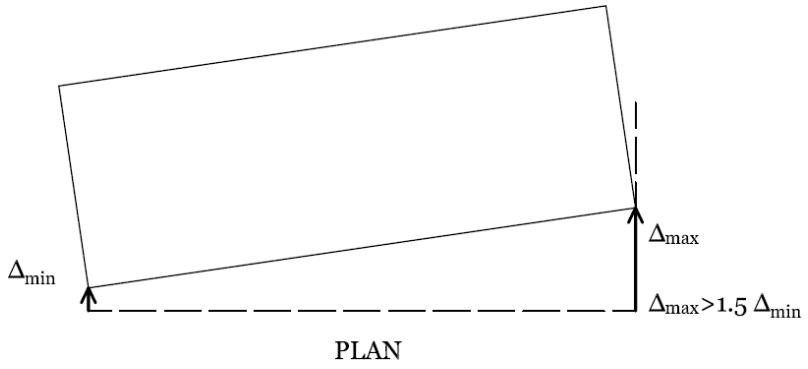


*Figure 5-2: **Torsion Irregularity***

5.5.2.2 Re-entrant Corners Irregularity

A structure is said to have re-entrant corner in a direction, if its structural configuration has a projection of greater than 15% of its overall dimension in that direction (Figure 5-3).

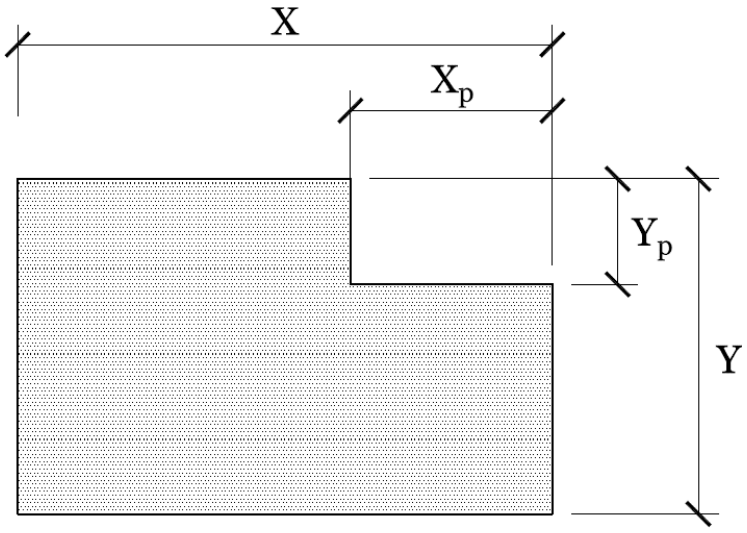


Figure 5-3: ***Re-entrant Corners***

5.5.2.3 Diaphragm Discontinuity Irregularity

Diaphragm discontinuity irregularity is considered to exist a diaphragm has a cutout or open area greater than 50% of the gross enclosed diaphragm area, or the effective diaphragm stiffness changes more than 50% from one story to the next (Figure 5-4).

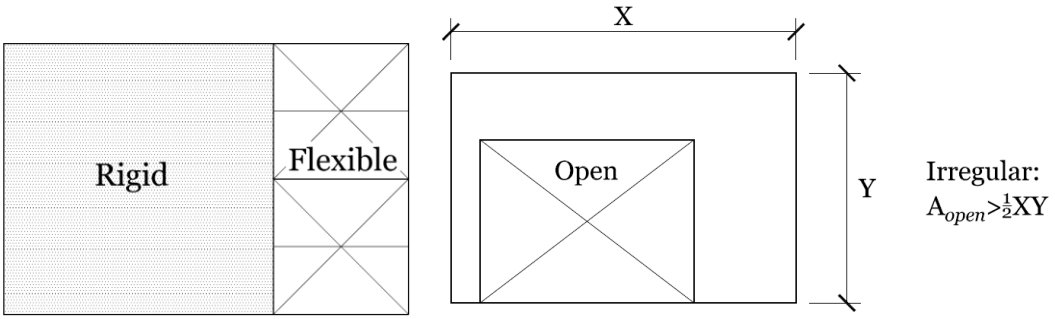


Figure 5-4: ***Diaphragm Discontinuity***

5.5.2.4 Out of plane offset Irregularity

Out of Plane offset irregularity is said to exist where there is a discontinuity in a lateral force resisting path, such as an out-of-plane of at least one vertical element (Figure 5-5).

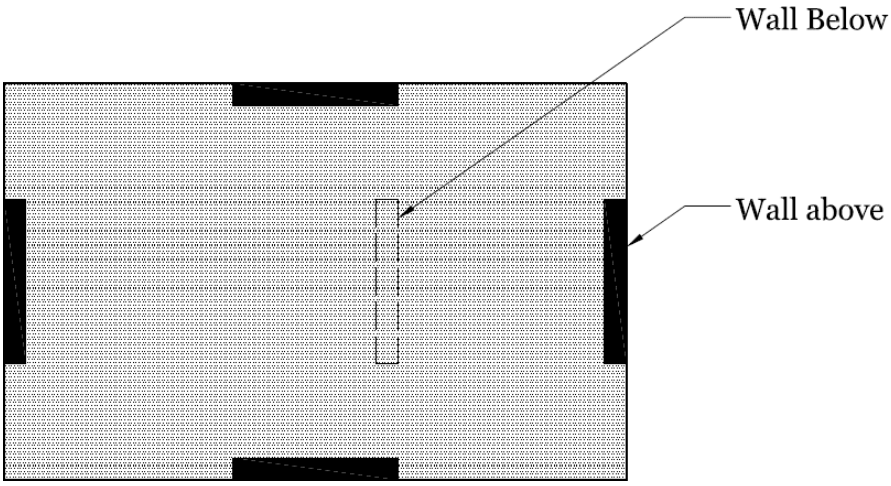


Figure 5-5: ***Out of plane offset***

5.6 DRIFTS AND DISPLACEMENTS

5.6.1 Determination of Design Horizontal Deflections

5.6.1.1 Ultimate limit state

The design horizontal deflections shall be determined by multiplying the horizontal deflection found from Equivalent Static Method or Modal Response Spectrum Method by the Ductility factor (R_μ).

5.6.1.2 Serviceability limit state

The design horizontal deflection for serviceability limit state shall be taken as equal to the horizontal deflections calculated either by Equivalent Static Method or Modal Response Spectrum Methods.

5.6.2 Building Separations

Parts of buildings or buildings on the same site which are not designed to act as an integral unit shall be separated from each other by a distance of not less than the sum of the design horizontal deflections determined in accordance with **5.6.1.**

Separation spaces shall be detailed and constructed to remain clear of debris and other obstructions. The width of such spaces shall allow for all constructional tolerances.

5.6.3 Inter-Story Deflections

The ratio of the inter-story deflection to the corresponding story height shall not exceed:

0.025 at ultimate limit state

0.006 at serviceability limit state

The deflections shall be obtained by using the effective stiffness properties of the components as given in 3.4.

5.7 ACCIDENTAL ECCENTRICITY

For the analysis for torsional effects, the applied torsion at each level shall use either the forces calculated by the Equivalent Static Method or the combined story inertial forces found in a Modal Response Spectrum Method. The accidental eccentricity can be taken as ±0.1b.

6 Equivalent Static Method

6.1 HORIZONTAL BASE SHEAR COEFFICIENT

6.1.1 Ultimate Limit State

For the ultimate limit state, the horizontal base shear coefficient (design coefficient), $C_d(T_1)$, shall be given by:

$$C_d(T_1) = \frac{C(T_1)}{R\mu \times \Omega u} \quad \text{.....6.1(1)}$$

Where,

C (T_1) = Elastic Site Spectra as per 4.1.1

R_μ = Ductility Factor as per 5.3

$\boldsymbol{\Omega}u$ = Over strength Factor for ULS as per 5.4

6.1.2 Serviceability Limit State

For the serviceability limit state, the horizontal base shear coefficient (design coefficient), $C_d(T_1)$, shall be given by:

$$C_d(T_1) = \frac{Cs(T_1)}{\Omega s} \text{.....6.1(2)}$$

Where,

$C_s(T_1)$ = Elastic Site Spectra determined for Serviceability Limit State as per 4.2

$\boldsymbol{\Omega}_s$ = Over strength Factor for SLS as per 5.4

6.2 HORIZONTAL SEISMIC BASE SHEAR

The horizontal seismic base shear, V, acting at the base of the structure, in the direction being considered, shall be calculated as:

$V = C_d(T_1)\, W$ ..... 6.2(1)

Where,

$C_d(T_1)$ = Horizontal base shear coefficient as per 6.1

W = Seismic Weight of the structure as per 5.2

6.3 VERTICAL DISTRIBUTION OF SEISMIC FORCES

The lateral seismic force (F_i) induced at each level 'i' shall be calculated as:

$$F_i = \frac{W_i h_i^k}{\sum_i^n W_i h_i^k} \text{ x } V \quad \text{.....6.3(1)}$$

Where,

W_i = seismic weight of the structure assigned to level 'i';

h_i= height (m) from the base to level 'i';

n= total number of floors/levels

V= horizontal seismic base shear calculated as per 6.2

k= an exponent related to the structural period as follows:

- for structure having time period T≤0.5sec, k=1
- for structure having time period T≥2.5sec, k=2
- for structure having period between 0.5 sec and 2.5 sec, k shall be determined by linear interpolation between 1 and 2.

6.4 POINTS OF APPLICATION OF EQUIVALENT STATIC FORCES

The equivalent static forces calculated as per 6.3 shall be assumed to act simultaneously at each level in the direction being considered and shall be applied through points eccentric to the center of mass at each level as per 5.7.

7 Modal Response Spectrum Method

7.1 ULTIMATE LIMIT STATE

For the ultimate limit state, the horizontal base shear co-efficient for each mode, $C_d(Ti)$, shall as given by:

$$C_d(T_i) = \frac{C(T_i)}{R\mu \times \Omega u} \quad \text{.....7.1(1)}$$

Where,

$C(T_i)$ = Elastic Site Spectra at period Ti as per 4.1.1

Ti = fundamental period of the ith mode of vibration

R_μ = Ductility Factor as per 5.3

$\boldsymbol{\Omega}u$ = Over strength Factor for ULS as per 5.4

7.2 CALCULATION OF BASE SHEAR FORCE FOR EACH MODE

$$V_i = C_d(T_i) \times W_i \quad \text{.....7.1(2)}$$

Where,

W_i = Effective modal gravity load of i^{th} mode of vibration

$$= \frac{\left[\sum_{j=1}^{n} w_j \emptyset_{ji}\right]^2}{\sum_{j=1}^{n} w_j \left[\emptyset_{ji}\right]^2}$$

w_j = portion of gravity load at story level j

ϕ_{ji} = mode shape coefficient at floor j in mode i

The modal force for i^{th} mode of vibration at each of the story level j is determined as follows:

$$F_{ji} = \frac{w_j \emptyset_{ji}}{\sum_{j=1}^{n} \emptyset_{ji} w_j} \times V_i \quad \text{.....7.1(3)}$$

where,

w_j = portion of gravity load at story level j

ϕ_{ji} = mode shape coefficient at floor j in mode i

7.3 NUMBER OF MODES TO BE CONSIDERED

A sufficient number of modes shall be included in the analysis to include at least 90% of the total seismic mass in the direction under consideration.

All modes that are not part of the horizontal load resisting systems shall be ignored in modal combination

The modal combination shall be carried out only for modes with natural frequency less than 33 Hz; the effect of modes with natural frequencies more than 33 Hz shall be included by the missing mass correction procedure following established principles of structural dynamics.

7.4 COMBINATION OF MODAL EFFECTS

a. The combination of modal effects (such as story shear, moment, drift, displacements) shall be carried out using an established method such as Square Root of the Sum of the Squares (**SRSS)** or the Complete Quadratic Combination (**CQC)** method or any other generally accepted combination methods.

b. Modes shall be considered to be closely spaced if their frequencies are within 15%. For such modes, if the SRSS combination method is used, the modal action effects from any modes shall be first combined by direct summation ignoring any signs.

7.5 SCALE FACTOR FOR DESIGN VALUES OF THE COMBINED RESPONSE

When the design base shear (V_R) obtained by combining the modal base shear forces is less than the base shear (V) calculated using Equivalent Static Method; the member forces, story shear forces & base reactions obtained from the MRS method shall be multiplied by V/V_R.

Where, V = Base Shear determined from Equivalent Static Method

V_R = Base Shear determined from Modal Combination

8 Elastic Time History Analysis

8.1.1 Structural Modeling Requirements

8.1.1.1 Modeling

Three dimensional models of the structure shall be required for carrying out the analysis. The analysis consists of an analysis of a linear mathematical model of the structure to obtain various response quantities employing the methods of numerical integration based on ground motion acceleration histories compatible with the design response spectrum for the site.

8.1.1.2 Gravity Load

Gravity loads calculated as per clause 5.2 shall be applied to the structural model.

8.1.1.3 P-Delta Effect

P-Delta effects shall be included in the analysis.

8.1.1.4 Torsion

Inherent eccentricity arising due to offset in center of mass from the center of rigidity at each level shall be included in the analysis.

8.1.1.5 Damping

Linear viscous damping shall not exceed 5%.

8.1.1.6 Below grade Structure elements

For structures having structures below grade such as basements, the structural model shall extend to the foundation level and ground motions shall be applied at the foundation level.

8.1.2 Ground Motions

The ground motion selection, scaling, application and analysis details shall be as prescribed in Clause 9.3.2.

8.1.3 Evaluation of response quantities

If less than 7 numbers of ground motion records are used, maximum values the response quantities from these ground motions shall be used. If the number of ground motions used is more than 7, then average values of the

considered number of ground motions shall be used for evaluation of response quantities.

8.1.3.1 Inter story drifts

The inter-story drift shall not exceed the limits given in clause 5.6.

8.1.3.2 Member strengths

For member strength check, the final values of member actions obtained from elastic time-history analysis shall be divided by the ULS ductility factor $R\mu$.

9 Non-linear Static and Dynamic Analysis

9.1 GENERAL

A Non-linear analysis shall consist of an analysis of a mathematical model of the structure that accounts for the strength of the materials and their post-elastic behavior.

9.2 NON-LINEAR STATIC ANALYSIS

Non-linear static analysis (also known as pushover analysis) is a procedure where a mathematical model incorporating the inelastic post yield behavior of the structural elements is subjected to monotonically increasing horizontal loads until target displacement is reached.

9.2.1 Modeling and Analysis

Seismic forces shall be applied in both positive and negative directions. Maximum seismic effects as a result of this application shall be used.

Gravity loads shall be applied to appropriate elements of the structural model. At least, a bilinear force-displacement relation shall be used at element level.

9.2.2 Load pattern

Load pattern based on the first mode shape shall be applied in the direction under consideration.

9.2.3 Control node

The control node shall be located at the center of mass at the roof of the building. The top of penthouse shall not be considered as the roof.

9.2.4 Capacity curve

A plot of base shear versus the control node displacement shall be established for control displacement values ranging from zero to 150 % of the target displacement.

9.2.5 Target displacement

The target displacement shall serve as an estimate of the global displacement of the structure which is expected to experience in an earthquake. It shall be defined as the seismic demand derived from the

elastic response spectrum in terms of the displacement of an equivalent single degree of freedom system. Target displacement can be determined using N2 Method or any other method established in literature.

9.3 NON-LINEAR TIME HISTORY ANALYSIS

Non-linear time history analysis shall be carried out through direct numerical integration of the differential equations of ground motion acceleration time histories. The numerical integration time history analysis may be used for all types of structures to verify that the specific response parameters are within the limits of acceptability assumed during design.

9.3.1 Structural Modeling Requirements

9.3.1.1 Modeling

Three dimensional models of the structure shall be required for carrying out the analysis. The structural model shall include the post elastic hysteretic behavior of elements and shall account for all the significant yield, strength degradation, stiffness degradation and hysteretic pinching.

At least, a bilinear force-displacement relation shall be used at element level.

9.3.1.2 Gravity Load

Gravity loads calculated as per clause 5.2 shall be applied to the structural model.

9.3.1.3 P-Delta Effect

P-Delta effects shall be included in the analysis.

9.3.1.4 Torsion

Inherent eccentricity arising due to offset in center of mass from the center of rigidity at each level shall be included in the analysis.

9.3.1.5 Damping

Hysteretic energy dissipation of structural elements shall be included in the analysis as well.

9.3.1.6 Below grade Structure elements

For structures having structures below grade such as basements, the structural model shall extend to the foundation level and ground motions shall be applied at the foundation level.

9.3.2 Ground Motions

9.3.2.1 Number of Ground Motions

A minimum of **three** ground motions shall be used in 2D time history analysis.

For 3D time history analyses conducted on torsionally sensitive structures, three pairs of orthogonal ground motions shall be used.

Appropriate ground motions shall be selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that seismic hazard at the design location.

Where the required number of recorded ground motions is not available, appropriate simulated ground motions shall be used to make up the total number required.

9.3.2.2 Scaling of Ground Motions

Following procedures shall be carried out for scaling the selected ground motions:

a) The elastic site spectra C(T) given in clause 4.2.1 divided by the ULS Overstrength factor (Ω_u) shall be used as the target spectrum for scaling the ground motions.

b) The selected ground motions shall be scaled to match the target spectrum between periods $\mathbf{T_n}$ **and** $\sqrt{\mathbf{R_\mu x T_1}}$, where T_1 is the fundamental period of vibration of the structure, T_n is the period of the highest vibration mode to ensure 90% mass participation and R_μ is the ULS ductility factor as per 5.3.1.

c) The Scaling factor for all ground motions shall be between 0.33 and 3. Where a selected ground motion does not match this criteria, it shall be discarded and a new ground motion shall be selected.

d) For 3D analyses, scaling factor for both orthogonal motions shall be determined and the lower value shall be used to scale both

components of the ground motion pair. Obviously, only one of the two scaled motions will match the target spectra and the other will be smaller.

9.3.2.3 Application of Ground Motions

a) The scaled ground motion/s shall be applied to the supports of the structural model.

b) For 3D analyses, the two ground motion components of each pair shall be applied in orthogonal directions along the principal axes of the building structure.

c) For each pair, two analyses shall be conducted by interchanging the directions of the ground motions.

9.3.2.4 Analysis time step

The analysis time step:

a) Shall not be greater than the step at which the records are digitized

b) Shall be less than or equal to:

- $T_1/100$
- T_n
- 0.01s

Where

T_1 = the fundamental translational period in the direction under consideration

T_n = Period of highest mode in the same direction required to achieve the 90% mass participation as described in modal analysis

9.3.3 Evaluation of response quantities

If less than 7 numbers of ground motion records are used, maximum values the response quantities from these ground motions shall be used. If the number of ground motions used is more than 7, then average values of the considered number of ground motions shall be used for evaluation of response quantities.

9.3.3.1 Inter story drifts

The inter-story drift shall not exceed the limits given in clause 5.6.

9.3.3.2 Member strengths

The inelastic deformation demands shall not exceed the limits given in appropriate material standards.

10 Parts and Components

10.1 GENERAL

This section specifies the minimum design requirements for non-structural components of architectural, mechanical and electrical systems, their support and connections.

All elements, components or equipment shall be positively connected to the structure to resist the specified seismic loads. Where friction due to gravity is used to provide the required resistance to horizontal seismic forces, a friction reduction factor of 0.5 shall be used in calculating the frictional resistance.

If the weight of the non-structural component is more than 25% of the seismic weight of the building, provisions of this section shall not be used.

10.2 SERVICE CUT-OFFS

If continued operation of a facility during strong seismic motions presents an excessive risk, an automatic shut-off system, which will operate at a pre-determined ground acceleration, not exceeding 0.2g, shall be provided. In such cases, all equipment required for safe shut-down shall be capable of resisting the shut-off level irrespective of other requirements of this Section.

10.3 DESIGN SEISMIC FORCE

All elements and components shall be designed for a design seismic force (F_p) along its principal direction. F_p shall be applied at the component's center of gravity and distributed relative to the component's mass distribution. The design seismic force (F_p) shall be calculated using following equation:

$$F_P = Z\left(1 + \frac{h_p}{H}\right)\frac{a_p}{\mu_p} I_p W_p \quad \text{.....10.3(1)}$$

Where,

Z = Seismic Zoning factor as per 4.1.4

a_p = Component amplification factor as per 10.3.1

μ_p = Component ductility factor as per 10.3.2

I_p = Component Importance factor as per 10.3.3

W_p = Component weight

h_p = height of attachment of the component

H = Total height of structure

10.3.1 Component Amplification Factor

Component amplification factor (a_p) represents the dynamic amplification of the component relative to the fundamental time period of the structure. Its value varies from 1 to 2.5. The component amplification factor (a_p) is stipulated in Table 10-1:

Table 10-1 *Component Amplification Factor*

Absolute difference between the natural periods of the building and the component $\|T_1-T_p\|$	**a_p**
$\|T_1-T_p\| >$ 0.5 sec	1.0
$\|T_1-T_p\| =$ 0 sec; (i.e.**$T_1=T_p$)**	2.5
0.5 sec **$> \|T_1-T_p\| >$** 0 Sec	To be linearly interpolated between 1.0 and 2.5

10.3.2 Component Ductility Factor

Component ductility factor (μ_p) represents the ductility and energy dissipation capacity of the components and its connections. Its value varies from 1.5 to 2.5. The component ductility factor (μ_p) is stipulated in Table 10-2. For any high deformability non-structural components and attachments, a higher value, not exceeding 3.5 may be used if/as supported by research

Table 10-2*: Component Ductility Factor*

Element Class	**μ_p**
Low deformability components and attachments (Examples include non-structural walls, brick chimneys and partitions)	1.5
Moderate deformability components and attachments (Examples include cantilevers, metal chimneys, parapets, signs, billboards)	2.5

10.3.3 Component Importance Factor

All parts and components shall be assigned to appropriate importance class as stipulated in Table 10-3:

***Table 10-3**: Component Importance Factor*

Element Class	I_p
Components containing hazardous contents	1.5
Component required to function for life-safety after an earthquake (Examples include fire protection sprinkler systems, egress stairways)	1.5
Storage facilities open to public	1.5
Components which are needed for continued operation of an emergency facility after an earthquake	1.5
All other components	1.0

10.4 OTHER REQUIREMENTS

In addition to the requirements stipulated in this section, following additional requirements shall also be considered:

a) Connections to ornamentations, veneers, appendages and exterior panels including anchor bolts, shall be corrosion-resisting and ductile, with adequate anchorages.

b) In the case of precast concrete panels, anchorages shall be attached to, or hooked around, panel reinforcing.

c) The seismic weight of containers and the like shall include the weight of the contents.

d) Hanging or swinging lights shall have a safety cable attached to the structure and the fixture, capable of supporting a lateral load equal to four times the weight.

e) The support systems for suspended ceilings shall be designed and constructed so as to avoid sudden or incremental failure or excessive deformations that would release ceiling components.

f) The non-structural component that significantly affects the structural response of the building should be treated as structural and relevant provisions for design of the structure should be applied.

g) Contents of museums and similar items of historical or artistic value that are non-functional items should be restrained against seismic forces. Special advice should be obtained for detailing such restraints.

ANNEX A: DESIGN AND DETAILING OF REINFORCED CONCRETE STRUCTURES FOR EARTHQUAKE LOADS

1 General

This section covers the requirements for designing and detailing of members of reinforced concrete (RC) structures designed to resist lateral effects of earthquake shaking, so as to give them adequate stiffness, strength and ductility to resist severe earthquake shaking without collapse. This standard addresses lateral load resisting structural systems of RC structures composed of,

a) RC moment resisting frames,
b) RC moment resisting frames with unreinforced masonry infill walls,
c) RC structural walls
d) a combination of RC moment resisting frames and RC structural walls

1.1 DEFINITIONS

For the purpose of this standard, the following definitions shall apply.

Beams: These are members (generally horizontal) of moment resisting frames which are subjected to flexural and shear actions.

Boundary Elements: These are portions along the ends of a structural wall that are strengthened and confined by longitudinal and transverse reinforcement. They may have the same thickness as that of the wall web.

Columns: These are members (generally vertical) of moment resisting frames which are subjected to a combination of axial, flexural and shear actions.

Concrete Cover: It is the concrete around the perimeter of the cross-section which is not confined by transverse reinforcement.

Crosstie: It is a continuous reinforcing bar having a 135° hook with an extension of 6 times diameter (but not < 65 mm) at one end and a hook not less than 90 degrees with at least a 6 times diameter at the other end. The hooks shall engage peripheral longitudinal bars. The 90-degree hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end.

Transverse Reinforcement: It is a continuous bar profile bent in square, rectangular, triangular or trapezoidal shape, which may also include straight cross ties. It ends with a 135° hook with an extension of 6 times diameter (but not < 65 mm) at one end and a hook not less than 90° with an extension of 6 times diameter (but not < 65 mm) at the other end. The hooks shall engage a number of peripheral longitudinal bars. In general, the 90° hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end. Transverse reinforcement is also called hoops and/or stirrups.

Gravity Columns in Buildings: It is a column, which is not part of the lateral load resisting system and designed only for actions (i.e., axial force, shear force and bending moments) due to gravity loads. But, it should be able to resist the gravity loads at lateral displacement imposed by the earthquake forces.

Lateral Force Resisting System: It is that part of the structural system which participates in resisting forces induced by earthquake.

1.2 NOTATIONS

For the purpose of this standard, the following letter notations shall have the meaning indicated against each; where other notations are used, they are explained at the appropriate place.

A_e	Effective cross sectional area of a joint
A_{ej}	Effective shear area of a joint
A_g	Gross cross-sectional area of column, wall
A_h	Horizontal shear reinforcement in walls
A_{cc}	Area of concrete core of column
A_{sd}	Reinforcement along each diagonal of coupling beam
A_{sh}	Area of cross section of bar forming spiral or link
A_{tr}	Area of transverse reinforcement
A_v	Vertical reinforcement in a joint
b_b	Width of beam
B_c, b_c	Width of column
b_j	Effective width of a joint
D	Overall depth of beam

D_k Diameter of column core measured to the outside of transverse reinforcement

d Effective depth of member

d_b Diameter of longitudinal bar

d_w Effective depth of wall section

E_s Elastic modulus of steel

f_{ck} Characteristic compressive strength of concrete cube

f_y Yield stress of steel reinforcing bars, or 0.2 percent proof strength of reinforcing steel whose yield point is not obvious

h Longer dimension of rectangular confining link measured to its outer face

h_c Depth of column

h_j Effective depth of a joint

h_{st} Clear story height

h_w Overall height of RC structural wall

L_{AB} Clear span of beam

L_d Development length of bar in tension

l_o Length of member over which special confining reinforcement is to be provided

L_w Horizontal length of wall/longer cross- section dimension of wall

L_s Clear span of couplings beam

M_u Design moment of resistance of entire RC beam, column or wall section

M_{ct} Design moment of resistance of top column at a joint

M_{cb} Design moment of resistance of bottom column at a joint

M_{bl} Design moment of resistance of left beam at a joint

M_{br} Design moment of resistance of right beam at a joint

M_u^{Ah} Hogging (negative) design moment of resistance of beam AB at end A

M_u^{As} Sagging (positive) design moment of resistance of beam AB at end A

M_u^{Bh} Hogging (negative) design moment of resistance of beam AB at end B

M_u^{Bs} Sagging (positive) design moment of resistance of beam AB at end B

M_u^{BL} Design moment of resistance of beam framing into column from left

M_u^{BR} Design moment of resistance of beam framing into column from right

M_{uw} Design moment of resistance of web of RC structural wall alone

P_u Factored axial load

s_v Spacing of transverse reinforcement along the length of a beam or column

t_w Thickness of the web of RC structural wall

$V_{u,a}^{D+L}$ Factored shear force demand at end A of beam AB due to dead and live loads

$V_{u,b}^{D+L}$ Factored shear force demand at end B of beam AB due to dead and live loads

V_j Design shear resistance of a joint

V_u Factored shear force

V_{us} Design shear resistance offered at a section by transverse reinforcement

α Inclination of diagonal reinforcement in coupling beam

ρ Area of longitudinal reinforcement as a fraction of effective area of cross-section in a RC beam, column or structural wall

ρ_c Area of longitudinal reinforcement on the compression face of a beam as a fraction of effective area of cross-section

ρ_{max} Maximum area of longitudinal reinforcement permitted on the tension face of a beam as a fraction of effective area of cross-section

ρ_{min} Minimum area of longitudinal reinforcement permitted on the tension face of a beam as a fraction of effective area of cross-section

τ_c Design shear strength of concrete

$\tau_{c,\,max}$ Maximum nominal shear stress permitted at a section of RC beam, column or structural wall

τ_v Nominal shear stress at a section of RC beam, column or structural wall

1.3 UNITS

All dimensions are in millimeter, loads in Newton and stresses in MPa, unless otherwise specified.

2 Materials

2.1 Grade of Material

Minimum grade of structural concrete shall be M20, but M25 for buildings more than 12 m in height.

Steel reinforcement used shall be of,

a) Grade Fe 415 or less; or
b) High strength deformed steel bars produced by thermo-mechanical treatment process having elongation capacity of more than 15 percent; e.g. Grade Fe 500 and Fe 550.

2.2 Expected Material Strength

The ratio of the actual ultimate strength to the actual yield strength or the actual 0.2 percent proof strength of the reinforcement shall be at least 1.15.

3 Location of Plastic Hinges

In moment resisting frames, the plastic hinges shall be normally located at the ends of the beam member. Such reversing plastic hinges yield in both sagging in hogging actions. In frames where gravity actions are significant in comparison to the seismic actions, unidirectional plastic hinges in beams may also form away from the column faces.

Plastic hinges are permitted to occur in columns at the following locations:

i. at the base of the frame
ii. at the top of the columns in upper story of multi-story buildings

In shear wall structures, the plastic hinges shall be primarily located at the base of the wall.

4 Moment Resisting Frames

4.1 BEAMS

Requirements of this section shall apply to beams resisting earthquake-induced effects, in which the factored axial compressive stress does not exceed $0.08f_{ck}$. Beams, in which the factored axial compressive stress exceeds $0.08f_{ck}$, shall be designed as per requirements of **4.2.**

4.1.1 Dimensional Limits

a) Beams shall preferably have width-to-depth ratio of more than 0.3
b) Beams shall not have width less than 200 mm
c) Beams shall not have depth D more than 1/4th of clear span.
d) Width of beam b_w shall not exceed the width of supporting member

4.1.2 Longitudinal Reinforcement

a) Beams shall have at least two 12 mm diameter bars each at the top and bottom faces.
b) Minimum longitudinal steel ratio ρ_{min} required on any face at any section is:

$$\rho_{min} = 0.24\frac{\sqrt{f_{ck}}}{f_y} \quad \text{.....4.1.2}$$

c) Maximum longitudinal steel ratio ρ_{max} provided on any face at any section is 0.025.
d) Longitudinal steel on bottom face of a beam framing into a column (at the face of the column) shall be at least half the steel on its top face at the same section. At exterior joints, the anchorage length calculation shall consider this bottom steel to be tension steel.
e) Longitudinal steel in beams at any section on top or bottom face shall be at least 1/4th of longitudinal steel provided at the top face of the beam at the face of the column; when the top longitudinal steel in the beam at the two supporting column faces is different, the larger of the two shall be considered.

4.1.3 Transverse Reinforcement

a) Only vertical links/stirrups shall be used in beams (see Fig. 4-1); inclined links/stirrups shall not be used.
b) Links/stirrups are permitted to be made of two pieces of bars also, namely a U-link with a 135° hook with an extension of 6 times diameter (but not less than 65 mm) at each end, embedded in the core concrete, and a cross-tie (see Fig. 4-1).
c) The hooks of the links/stirrups and cross-ties shall engage around peripheral longitudinal bars. Consecutive crossties engaging the same longitudinal bars

shall have their 90° hooks at opposite sides of the beam. When the longitudinal reinforcement bars are secured by cross-ties in beams that have a slab on one side, the 90° hooks of the cross-ties shall be placed on that side.

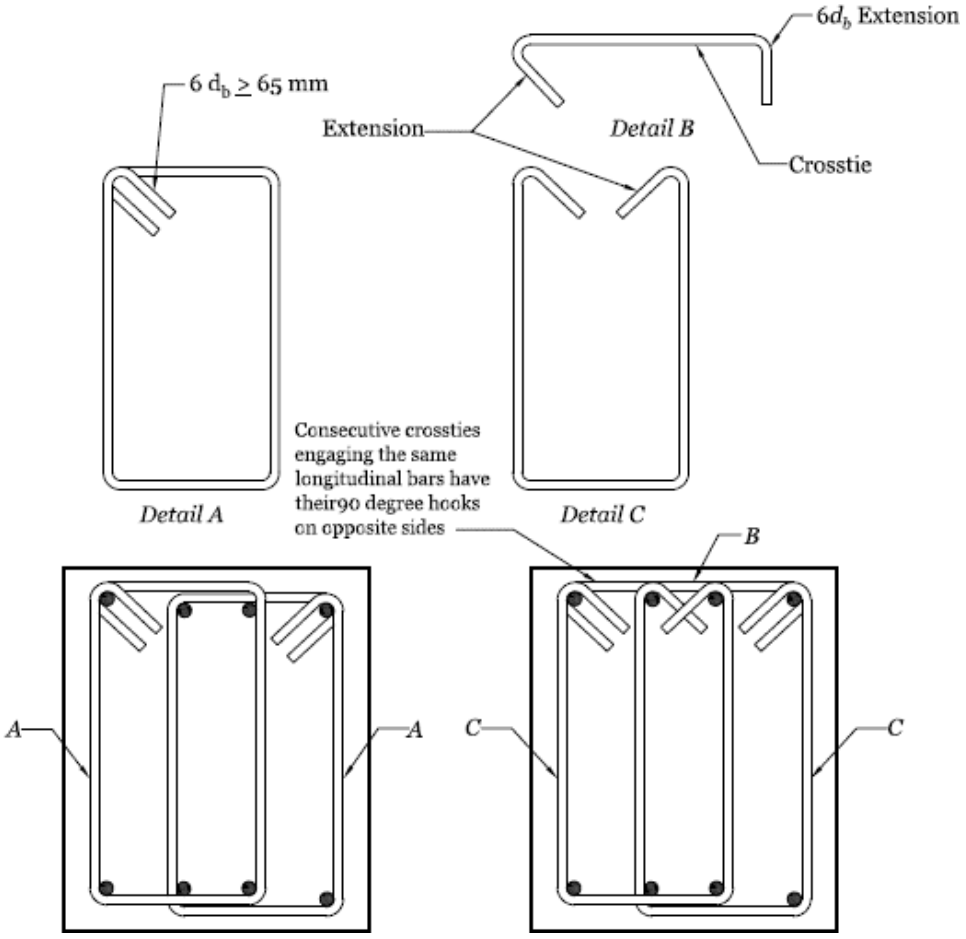


Figure 4-1 Details of Transverse reinforcement in beams

d) The minimum diameter of a transverse reinforcement shall be 8 mm.

e) Transverse reinforcement shall be designed to resist the shear resulting from larger of:

i. Factored shear force as per linear structural analysis; and

ii. Factored gravity shear force, plus equilibrium shear force when plastic hinges are formed at both ends of the beam (see Fig. 4-2) given by:

For sway to right:

$$V_{u,a} = V_{u,a}^{D+L} - 1.4\frac{(M_u^{As}+M_u^{Bh})}{L_{AB}} \quad \text{.....4.1.3.1}$$

$$V_{u,b} = V_{u,b}^{D+L} + 1.4\frac{(M_u^{As}+M_u^{Bh})}{L_{AB}} \quad \text{.....4.1.3.2}$$

For sway to left:

$$V_{u,a} = V_{u,a}^{D+L} - 1.4\frac{(M_u^{Ah}+M_u^{Bs})}{L_{AB}} \quad \text{.....4.1.3.3}$$

$$V_{u,b} = V_{u,b}^{D+L} + 1.4\frac{(M_u^{Ah}+M_u^{Bs})}{L_{AB}} \quad \text{.....4.1.3.}$$

Where M_u^{As}, M_u^{Ah}, M_u^{Bs} & M_u^{Bh} are sagging and hogging moments of resistance of the beam section at ends A and B, respectively. L_{AB} is clear span of the beam. $V_{u,a}^{D+L}$ and $V_{u,b}^{D+L}$ are the factored shear forces at ends A and B respectively, due to vertical loads acting on the span with load combination factors of 1 for dead load and 0.5 for live load . The beam shall be considered to be simply supported for this derivation.

The design shear force demand at end A of the beam shall be the larger of the two values of $V_{u,a}$ computed above. Similarly, the design shear force demand at end B shall be the larger of the two values of $V_{u,b}$ computed above.

f) Closely spaced links shall be provided over a length equal to 2d on either side of a section where flexural yielding may occur under earthquake effects. Over the remaining length of the beam, vertical links shall be provided at a spacing not exceeding d/2.

g) Spacing of links over a length of 2d at either end of a beam shall not exceed the minimum of (d/4; $8d_b$; 100 mm); however, it need not be less than 100mm.

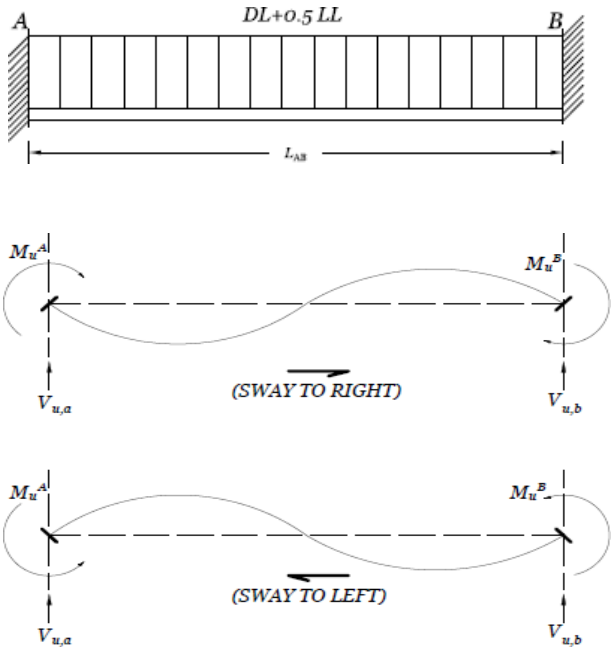


Figure 4-2 Calculation of Shear force demand in beams

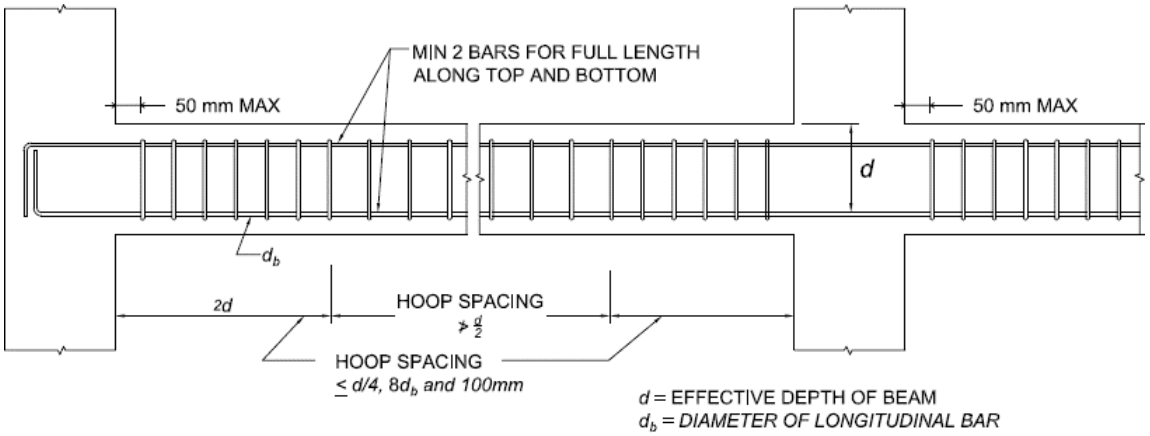


Figure 4-3 Details of transverse reinforcement in beams

h) The first link shall be at a distance not exceeding 50 mm from the joint face.

4.2 COLUMNS

Requirements of this section shall apply to columns resisting earthquake-induced effects, in which the factored axial compressive stress due to gravity and earthquake effects exceeds $0.08f_{ck}$.

4.2.1 Dimensional Limits

a) The minimum dimension of a column shall not be less than 20 d_b, where d_b is diameter of the largest diameter of longitudinal reinforcement bar in the beam passing through or anchoring into the column at the joint.
b) The minimum dimension of column shall be 300 mm.

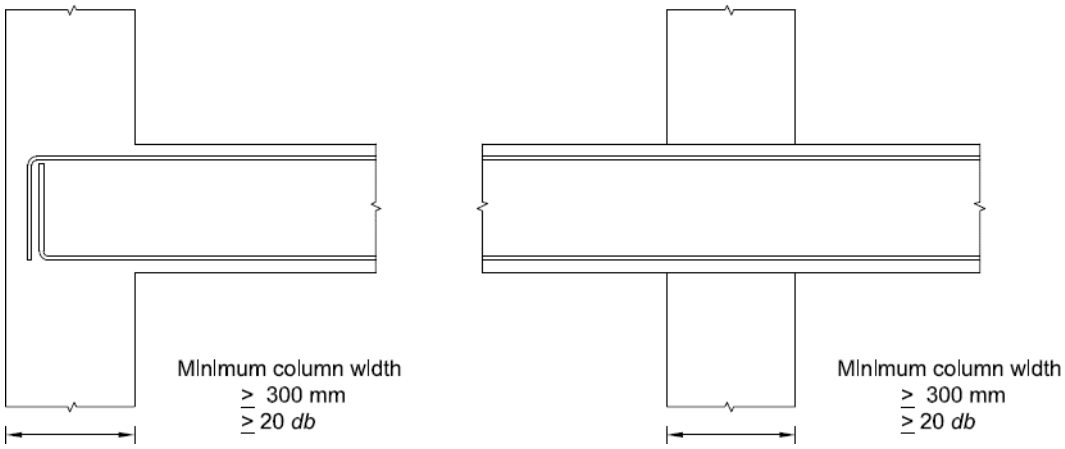


Figure 4-4 Requirement for minimum size of column member

c) Columns shall preferably have width-to-depth ratio of more than 0.45.

4.2.2 Longitudinal Reinforcement

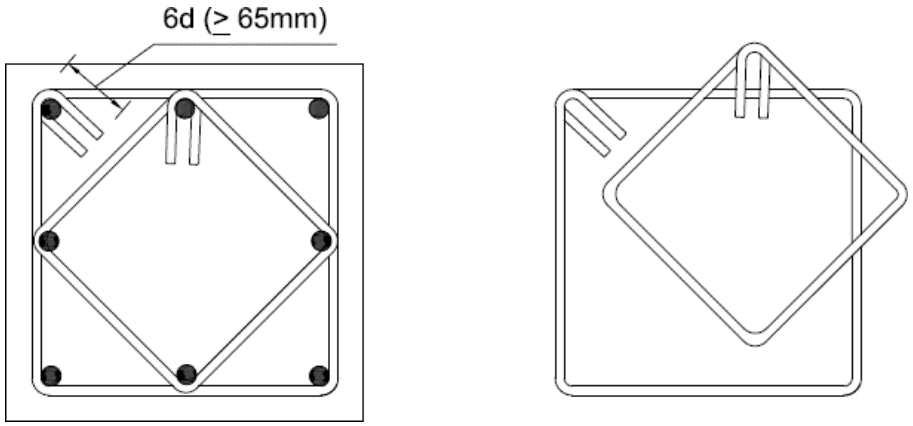
a) Circular columns shall have a minimum of 6 numbers of bars and rectangular columns shall have a minimum of 8 numbers of bars.
b) Minimum longitudinal steel ratio ρ_{min} shall be 0.01.
c) Maximum longitudinal steel ratio ρ_{max} shall be 0.04.
d) Minimum diameter of the longitudinal bar shall be 12 mm.

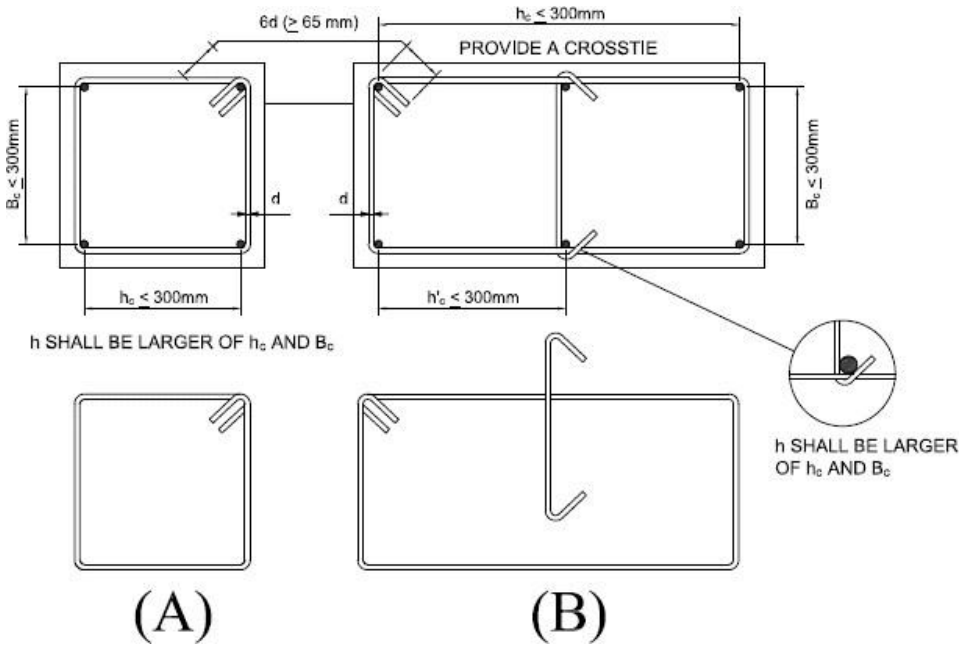
4.2.3 Transverse Reinforcement

a) Transverse reinforcement shall consist of closed loops of spiral/ circular links/hoops for circular columns, and rectangular links/hoops for rectangular columns.
b) The closed links/hoops shall have 135° hook ends with an extension of 6 times its diameter (but not < 65 mm) at each end, which are embedded in the confined core of the column (see Fig. 4-5).
c) The minimum diameter of a link/hoop shall be 8 mm
d) In rectangular columns, the maximum spacing of parallel legs of the links/hoops shall be 300 mm center to center. A cross-tie shall be provided, if the length of any side of the link/hoop exceeds 300 mm (see Fig. 4-5); the cross-tie shall be placed perpendicular to this link/hoop whose length exceeds 300 mm. Alternatively, a pair of overlapping links/hoops may be

provided within the column (see Fig. 4-5). In either case, the hook ends of the links and cross-ties shall engage around peripheral longitudinal bars. Consecutive cross-ties engaging the same longitudinal bars shall have their 90° hooks on opposite sides of the column. Crossties of the same or smaller bar size as the hoops shall be permitted; and,

e) The maximum spacing of links/hoops shall be half the least lateral dimension of the column.





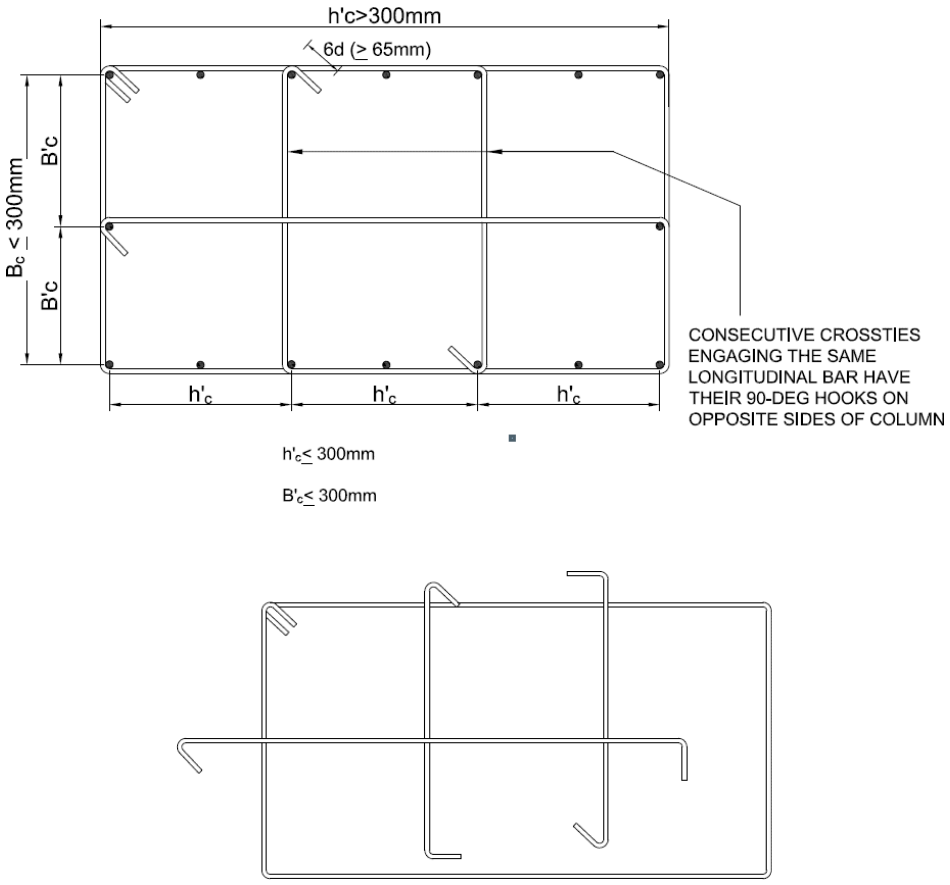


Figure 4-5 Details of transverse reinforcement arrangement in columns

f) Transverse reinforcement shall be designed to resist the shear resulting from larger of:

i. Factored shear force as per linear structural analysis; and

ii. Factored equilibrium shear force demand when plastic hinges are formed at both ends of the beams given by:

For sway to right:

$$V_u = \frac{1.4(M_u^{As} + M_u^{Bh})}{h_{st}} \quad \text{.....4.2.3.1}$$

For sway to left:

$$V_u = \frac{1.4(M_u^{Ah} + M_u^{Bs})}{h_{st}} \quad \text{.....4.2.3.2}$$

Where M_u^{As}, M_u^{Ah} , M_u^{Bs} and M_u^{Bh} are design sagging and hogging moments of resistance of beams framing into the column on opposite faces A and B, respectively, with one hogging moment and the other sagging (see Fig. 4-6); and h_{st} the story height.

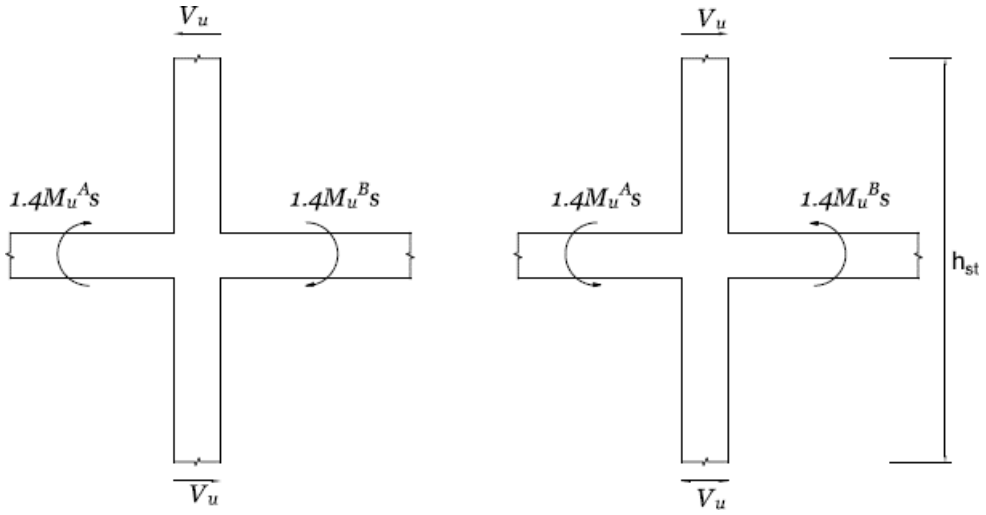


Figure 4-6 Calculation of Shear force demand in columns

4.3 SPECIAL CONFINING REINFORCEMENT

The amount of special confining reinforcement required in columns shall be calculated as follows:

a) The special confining reinforcement shall be provided over a length l_o from the face of the joint towards mid heights of columns on either side of the joint or at section where flexural yielding is likely to occur. The required length l_o calculated as follows:

$$l_0 = maximum\left(D_c; {}^{l_{cl}}/_{6}\ ; 450\ mm\right) \quad \text{.....4.3.1}$$

Where,

D_c = Largest cross sectional dimension of column

l_{cl} = Clear length of column

b) The spacing of the confining reinforcement shall be as follows:

$$Spacing\ = minimum\left({}^{1}/_{4}\,B_c; 6d_b; 100\ mm\right)$$

Where,

B_c = Smallest cross sectional dimension of column

d_b = Diameter of smallest longitudinal bar of column

c) The area, $A_{sh,}$ of cross section of the bar forming links/hoops or spiral shall be at least:

i. For circular links/hoops or spirals:

$$A_{sh} = Maximum\ of \begin{cases} 0.09 s_v D_k \frac{f_{ck}}{f_y}\left(\frac{A_g}{A_{cc}} - 1\right) \\ 0.024 s_v D_k \frac{f_{ck}}{f_y} \end{cases} \quad \text{.....4.3.2}$$

Where

S_v = pitch of spiral or spacing of links,

D_k = diameter of core of circular column measured to outside of spiral/link,

f_{ck} = characteristic compressive strength of concrete cube,

f_y = 0.2 percent proof strength of transverse steel reinforcement bars,

A_g = gross area of column cross-section, and

A_{cc} = area of concrete core of column = $\pi D_k^2/4$

ii. in rectangular links:

$$A_{sh} = Maximum\ of \begin{cases} 0.18 s_v h \frac{f_{ck}}{f_y}\left(\frac{A_g}{A_{cc}} - 1\right) \\ 0.05 s_v h \frac{f_{ck}}{f_y} \end{cases} \quad \text{.....4.3.3}$$

Where,

h = longer dimension of rectangular link/hoop measured to its outer face, which does not exceed 300 mm (see Fig 4-5), and

A_{cc} = area of confined concrete core in rectangular link/hoop measured to its outer dimensions. A_{cc} shall be measured as overall core area, regardless of link/hoop arrangement.

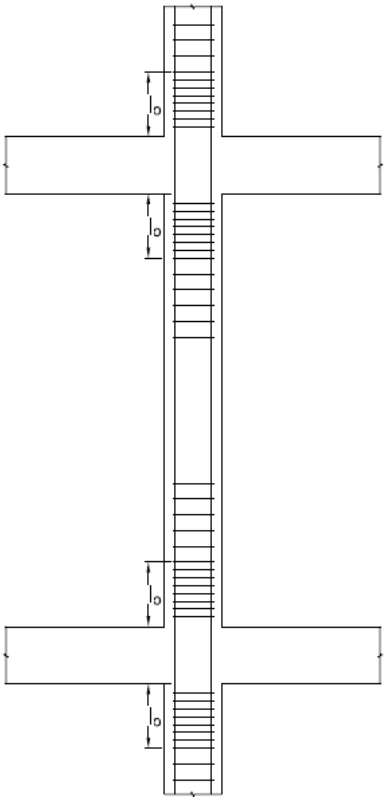


Figure 4-7 Confining stirrups in critical length l_o of column

d) When a column terminates into a footing or mat, special confining reinforcement shall extend at least 300 mm into the footing or mat (see Fig. 4-8).

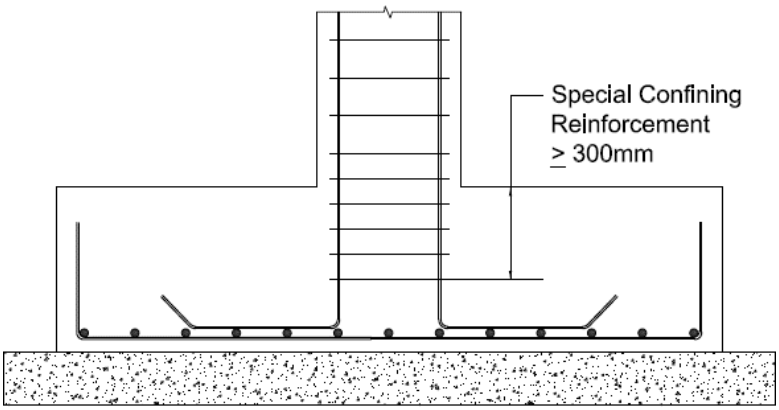


Figure 4-8 Confining reinforcement requirement in foundation

e) Special confining reinforcement shall be provided over the full height of a column which has significant variation in stiffness along its height.

f) Columns supporting reactions from discontinued stiff members, such as walls, shall be provided with special confining reinforcement over their full height.

4.4 BEAM-COLUMN JOINTS

4.4.1 Design of Beam-Column Joint for Distortional Shear

4.4.1.1 Nominal Joint Shear Strength

The nominal shear strength of a beam-column joint, V_{jc} is calculated as follows:

$$V_{jc} = \begin{cases} 1.5\, A_{ej}\sqrt{f_{ck}}\ for\ joints\ confined\ by\ beams\ on\ all\ four\ faces \\ 1.2\, A_{ej}\sqrt{f_{ck}}\ for\ joints\ confined\ by\ beams\ on\ three\ faces \\ 1.0\, A_{ej}\sqrt{f_{ck}}\ for\ other\ joints \end{cases} \quad ...4.4.1.1$$

where

A_{ej} = effective shear area of joint = $b_j w_j$,

b_j = effective breadth of joint perpendicular to the direction of shear force; and

w_j = effective width of joint along the direction of shear force.

The effective width of joint b_j (see Fig. 4-9) shall be obtained from following:

$$min(b_b; b_c)$$

Where

b_b = width of beam

b_c = width of column

h_c = depth of column in considered direction

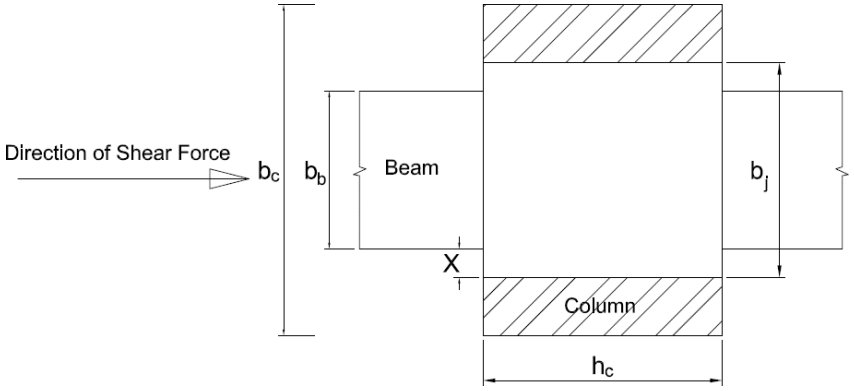


Figure 4-9 Effective joint width

4.4.1.2 Horizontal Joint Shear

Horizontal Joint shear in each principal direction in a joint is calculated as follows:

For Interior joint

$$V_{jh} = 1.25 f_y (A_{st1} + A_{st2}) - V_{col} \quad \text{.....4.4.1.2 (a)}$$

For Exterior joint

$$V_{jh} = 1.25 f_y A_{st1} - V_{col} \quad \text{.....4.4.1.2 (b)}$$

Where,

f_y = Yield strength of steel

V_{col} = Column Shear

A_{st1} = Total area of beam top reinforcement

A_{st2} = Total area of beam bottom reinforcement

4.4.1.3 Transverse Reinforcement in Joints

a) When $V_{jc} > V_{jh}$

No calculation for horizontal joint reinforcement is needed. However, Special confining reinforcement calculated as per clause 4.3 shall be provided through the joint within the depth of the shallowest beam framing into it.

b) When $V_{jc} < V_{jh}$

The amount of horizontal joint reinforcement is calculated as follows:

$$A_{jh} = \left(\frac{V_{jh} - V_{jc}}{f_y}\right) \quad \text{.....4.4.1.}$$

This calculated amount of horizontal joint reinforcement, however, shall not be less than the confinement reinforcement calculated as per clause 4.3. This amount of horizontal reinforcement shall be distributed uniformly within the joint.

4.4.2 Anchorage of Beam Longitudinal bars

At an exterior joint, top and bottom longitudinal bars of beams shall be provided with horizontal length beyond the inner face of the column, equal to horizontal development length (l_{dh}) terminating in a standard hook of length $12d_b$.The hook shall be located within the confined core of a column or of a boundary element with hook bent into the joint.

The value of l_{dh} is given by,

$$l_{dh} = \frac{f_y d_b}{4.85\sqrt{f_{ck}}} \quad \text{.....4.4.2(a)}$$

but ,

$$l_{dh} \nless D_c - Concrete\ Cover \quad \text{.....4.4.2(b)}$$

Where

d_b = diameter of largest longitudinal bar in beam in mm

f_y = Yield strength of steel

f_{ck} = Characteristic compressive strength of concrete

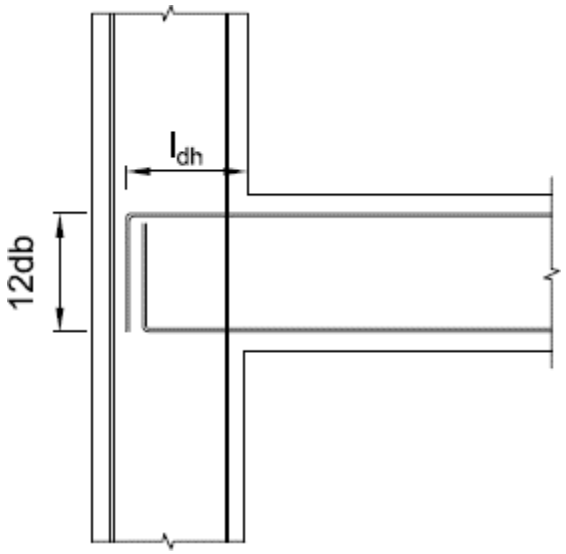


Figure 4-10 Anchorage of beam longitudinal bars in column

If the requirements for l_{dh} cannot be satisfied in the exterior beam-column joint, the beam can be extended horizontally in the form of exterior stubs (see figure 4-11)

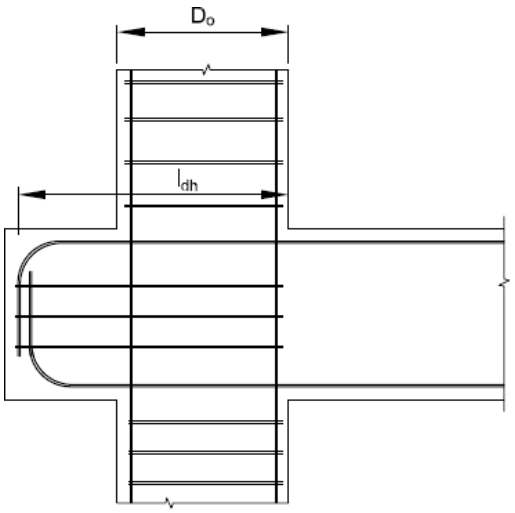


Figure 4-11 Provision of Beam stub

4.4.3 Development Length of Straight Deformed bars in tension

A bar should extend beyond the section at which it may be required to develop its strength f_y at least a distance not less than mentioned in **Table 4.3**.

Table 4.3: $L_d/\emptyset$ values for fully stressed bars in tension

Grade of Steel	Grade of Concrete				
	M 20	M 25	M 30	M 35	M40 and above
Fe 250	45	39	36	32	29
Fe 415	47	40	38	33	30
Fe 500	57	49	45	40	36

4.4.4 Column-Beam Moment Capacity Ratio

At every beam column junction in a frame, the summation of the moment capacities of the column end sections shall be greater than 1.2 times the summation of the beam end moment capacities.

$$\frac{\sum M_c}{\sum M_b} > 1.2 \quad \text{.....4.4.4}$$

Where,

ΣM_c = sum of the design moment of resistance of the column above and below the beam centerline;

ΣM_b = sum of the design moment of resistance of beams at the intersection of the beam and column centerlines

4.5 SPLICING OF BARS

4.5.1 Lap Splices

When longitudinal steel bars have to be continued for spans larger than their manufactured lengths, the bars shall be connected by either splicing or mechanical couplers.

a) lap splices shall not be provided,
   i. within a joint ;

ii. Within a distance of 2d from face of the column/ beam;
iii. Within a quarter length of the beam adjoining the location where flexural yielding may occur under earthquake effects.

b) Lap splices shall be provided only in the central half of clear column height

c) Not more than 50 percent of area of steel bars shall be spliced at any one section.

d) Closed links/stirrups/hoops shall be provided over the entire length of the beam/column over which the longitudinal bars are spliced.

e) The lap length shall not be less than the development length of the largest longitudinal reinforcement bar in tension.

f) The spacing of these links shall not exceed 150 mm.

g) The required area of transverse reinforcement A_{tr} within the lap zone of the longitudinal reinforcement of columns as well as beams spliced at the same location shall be calculated as follows:

$$A_{tr} = s\left(\frac{d_{bl}}{50}\right)\left(\frac{f_{yl}}{f_{yw}}\right) \quad \text{.....4.5.1}$$

where

A_{tr} = area of one leg of the transverse reinforcement

d_{bl} = diameter of the spliced bar

s = Spacing of the transverse reinforcement

f_{yl} = yield strength of the longitudinal reinforcement;

f_{yw} = yield strength of the transverse reinforcement

h) Splicing by use of mechanical couplers shall be permitted. Where mechanical couplers are preferred to connect the bars, the couplers shall confirm to contemporary international codes/standards/practice. Where established codes are not available to ascertain the adequacy of the couplers, they shall be verified by appropriate testing under conditions compatible with the required ductility.

i) Splicing by welding is not permitted within the critical regions of structural members where yielding of reinforcement is likely to take place. Welding of links, ties, inserts or other similar elements to vertical reinforcement bars required as per design is not permitted.

5 SHEAR WALLS

5.1 GENERAL REQUIREMENTS

1. The requirements of this section apply to shear walls that are part of lateral force resisting system of earthquake-resistant RC buildings.
2. The minimum thickness of special shear walls shall not be less than,
    a. 150 mm; and
    b. 300 mm for buildings with coupled shear walls.
3. The minimum ratio of length of a wall to its thickness shall be 4.
4. Shear walls shall be provided with uniformly spaced reinforcement in its cross-section along vertical and horizontal directions. The minimum reinforcement ratio shall be 0.0025 of the gross area in each direction.
5. Reinforcement bars shall be provided in two curtains within the cross-section of the wall, with each curtain having bars running along vertical and horizontal directions, when
6. The factored shear stress demand in the wall exceeds $0.25\sqrt{f_{ck}}$ MPa; or
7. The wall thickness is 200 mm or higher.
8. The largest diameter of longitudinal steel bars used in any part of a wall shall not exceed 1/10th of the thickness of that part.
9. The maximum spacing of vertical or horizontal reinforcement shall not exceed smaller of,
    a. $1/5^{th}$ of the horizontal length l_w of wall;
    b. 3 times the thickness t_w of the web of wall; and
    c. 450 mm.
10. Shear walls shall be founded on properly designed foundations and shall not be discontinued to rest on beams, columns or inclined members.

5.2 DESIGN FOR SHEAR FORCE

1. Nominal shear stress demand τ_v on a wall shall be estimated as:

$$\tau_v = \frac{V_u}{t_w d_w} \quad \text{.....5.2.1}$$

Where

V_u = factored shear force,

t_w = thickness of the web,

d_w = effective depth of wall section (along the length of the wall), which may be taken as 0.8 l_w for rectangular sections.

2. Design shear strength τ_c of concrete shall be calculated as follows:

$$\tau_c = \frac{0.85\sqrt{(0.8f_{ck})}\left(\sqrt{1+5\beta}-1\right)}{6\beta} \quad \text{.....5.2.2}$$

Where,

$$\beta = \begin{cases} \frac{0.8f_{ck}}{6.89p_t} \\ 1 \end{cases}$$, Whichever is greater

p_t = percentage of vertical reinforcement

f_{ck} = Characteristic compressive strength of concrete

3. Maximum design shear strength, $\tau_{c,max}$, shall be calculated as follows:

$$\tau_{c,max} = 0.62\sqrt{f_{ck}} \quad \text{.....5.2.3}$$

4. If $\tau_v > \tau_{c,max}$ the wall section shall be re-designed. If $\tau_v < \tau_{c,max}$, horizontal shear reinforcement shall be provided, whose area A_h is given by:

$$A_h = \frac{V_u - \tau_c t_w d_w}{0.87 f_y \left(\frac{d_w}{s_v}\right)} \quad \text{.....5.2.4}$$

However, the calculated reinforcement shall not be less than the minimum area specified in clause 5.1 (4).

5.3 BOUNDARY ELEMENTS

Boundary elements are portions along the wall edges that are strengthened by longitudinal and transverse reinforcement even if they have the same thickness as that of the wall web.

1. Boundary elements shall be provided in two opposite sides of walls, when the extreme fiber compressive stress in the wall occurs.
2. The length of boundary zone in each side shall be the maximum of 2 times the wall thickness and 0.2 times the wall length.
3. A boundary element shall have adequate axial load carrying capacity, assuming short column action, so as to enable it to carry axial compression arising from factored gravity load and lateral seismic shaking effects. The axial compression arising from lateral seismic effects may be calculated as:

$$\frac{M_u - M_{uv}}{C_w} \quad \text{.....5.3.3}$$

Where,

M_u = factored design moment on the entire wall section,

M_{uv} = moment of resistance provided by distributed vertical reinforcement across the wall section, and

C_w = center to center distance between the boundary elements along the two vertical edges of the wall.

4. The vertical reinforcement in the boundary elements shall not be less than 0.8 percent and not greater than 6 percent; the practical upper limit would be 4 percent to avoid congestion.
5. Boundary elements, where required shall be provided with special confining reinforcement as per clause 4.3.

5.4 DESIGN FOR AXIAL FORCE AND BENDING MOMENT

Design moment of resistance M_u of the wall section subjected to combined bending moment and compressive axial load shall be estimated in accordance with requirements of limit state design method, using the principles of mechanics involving equilibrium equations, strain compatibility conditions and constitutive laws.

5.5 COUPLING BEAMS

1. Coplanar special structural walls may be connected by means of coupling beams.
2. If earthquake induced shear stress τ_{ve} in coupling beam exceeds

$$\tau_{ve} > 0.1\sqrt{f_{ck}}\left(\frac{L_s}{D}\right);$$ .....5.5.2(a)

The entire earthquake-induced shear, bending moment and axial compression shall be resisted by diagonal reinforcement alone. Here,

L_s = clear span of coupling beam

D = overall depth

a) Area of this diagonal reinforcement along each diagonal shall be estimated as:

$$A_{sd} = \frac{V_u}{1.74\, f_y\, Sin\alpha}$$ .....5.5.2(b)

Where V_u is factored shear force on the coupling beam and α is the angle made by diagonal reinforcement with the horizontal.

b) At least 4 bars of 8 mm diameter shall be provided along each diagonal. All longitudinal bars along each diagonal shall be enclosed by special confining transverse reinforcement as per **4.3** at a spacing not exceeding 100 mm.

3. The diagonal of a coupling beam shall be anchored in the adjacent walls with an anchorage length of 1.5 times the development length in tension (see Fig. 5-1).

5.6 OPENINGS IN WALLS

1. Shear strength of a wall with openings should be checked at critical horizontal planes passing through openings.
2. Additional steel reinforcement shall be provided along all four edges of openings in walls. Further,

a) The area of these vertical and horizontal steel should be equal to that of the respective interrupted bars, provided half on either side of the wall in each direction.

b) These vertical bars should extend for full height of the story in which this opening is present.

c) The horizontal bars should be provided with development length in tension beyond the edge of the opening.

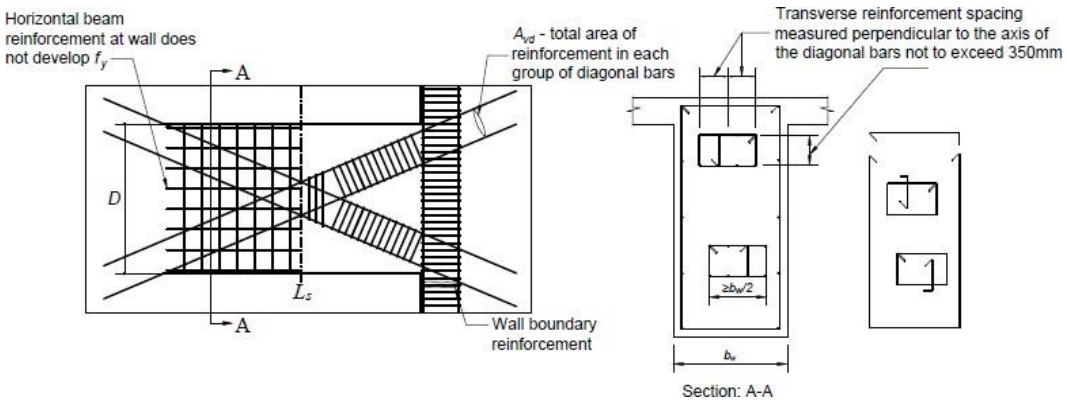

Horizontal beam reinforcement at wall does not develop f_y
A
D
L_s
A
A_{vd} - total area of reinforcement in each group of diagonal bars
Wall boundary reinforcement
Transverse reinforcement spacing measured perpendicular to the axis of the diagonal bars not to exceed 350mm
≥b_w/2
b_w
Section: A-A

Figure 5-1 Coupling beams with diagonal reinforcement

5.7 CONSTRUCTION JOINTS

Vertical reinforcement shall be provided across a horizontal construction joint. These reinforcements shall have an area, A_{st}, given by:

$$\frac{A_{st}}{A_g} \geq \frac{0.92}{f_y}\left(\tau_v - \frac{P_u}{A_g}\right) \quad \text{.....5.7}$$

Where

τ_v = factored shear stress at the joint

P_u = factored axial force (positive for compression),

A_g = gross cross-sectional area of the joint.

5.8 ANCHORAGE OF LONGITUDINAL BARS

Horizontal reinforcement shall be anchored near the edges of wall or in confined core of boundary elements.

5.9 SPLICING OF BARS

Splicing of the bars shall be done as per clause 4.5.

ANNEX B: DESIGN AND DETAILING OF STEEL STRUCTURES FOR EARTHQUAKE LOADS

1 General

This section covers the requirements for design and detailing of steel structures designed to resist the effects of earthquake forces so as to give them adequate strength, stability and ductility to resist severe earthquakes without collapse. Structures, which form a part of the gravity load resisting system but are not intended to resist the lateral earthquake loads, need not satisfy the requirements of this section, provided they can accommodate the resulting deformation without premature failure.

The provisions of this section are applicable to buildings in general. The provisions are applicable to following structural systems only:

1. Steel Moment Resisting Frames
2. Frames with concentric braces
3. Frames with eccentric braces

1.1 DEFINITIONS

Action Effect or Load Effect: The internal force, axial, shear, bending or twisting moment, due to external actions and temperature loads.

Action: The primary cause for stress or deformations in a structure such as dead, live, wind, seismic or temperature loads.

Beam: A member subjected predominantly to bending.

Braced Member: A member in which the relative transverse displacement is effectively prevented by bracing.

Buckling Load: The load at which an element, a member or a structure as a whole, either collapses in service or buckles in a load test and develops excessive lateral (out of plane) deformation or instability.

Buckling Strength or Resistance: Force or moment, which a member can withstand without buckling.

Built-up Section: A member fabricated by interconnecting more than one element to form a compound section acting as a single member.

Column: A member in upright (vertical) position which supports a roof or floor system and predominantly subjected to compression.

Compact Section: A cross-section, which can develop plastic moment, but has inadequate plastic rotation capacity needed for formation of a plastic collapse mechanism of the member or structure.

Dead Loads: The self-weights of all permanent constructions and installations including the self-weight of all walls, partitions, floors, roofs, and other permanent fixtures acting on a member.

Doubler Plates: Component object that represents a structure that strengthens a steel beam web or steel column web by thickening it. These plates act as reinforcement for the column web to prevent panel zone shear failure.

Effective Length: Actual length of a member between points of effective restraint or between effective restraint and free end, multiplied by a factor to account for the end conditions in buckling strength calculations.

Gravity Load: Loads arising due to gravitational effects.

Gusset Plate: The plate to which the members intersecting at a joint are connected.

Imposed (Live) Load: The load assumed to be produced by the intended use or occupancy including distributed, concentrated, impact, vibration and snow loads but excluding, wind, earthquake and temperature loads

Main Member: A structural member, which is primarily responsible for carrying and distributing the applied load or action.

Plastic Design: Design against the limit state of plastic collapse.

Plastic Hinge: A yielding zone with significant inelastic rotation, which forms in a member, when the plastic moment is reached at a section.

Plastic Moment: Moment capacity of a cross-section when the entire cross-section has yielded due to bending moment.

Plastic Section: Cross-section, which can develop a plastic hinge and sustain plastic moment over sufficient plastic rotation required for formation of plastic failure mechanism of the member or structure.

Rotation: The change in angle at a joint between the original orientation of two linear member and their final position under loading.

Secondary Member: Member which is provided for overall stability and/or for restraining the main members from buckling or similar modes of failure.

Semi-compact Section: Cross-section, which can attain the yield moment, but not the plastic moment before failure by plate buckling.

Shear Force: The in-plane force at any transverse cross-section of a straight member of a column or beam.

Shear Stress: The stress component acting parallel to a face, plane or cross-section.

Slender Section: Cross-section in which the elements buckle locally before reaching yield moment.

Slenderness Ratio: The ratio of the effective length of a member to the radius of gyration of the cross-section about the axis under consideration.

Stiffener: An element used to restrain or prevent the out-of-plane deformations of plates.

Strain: Deformation per unit length or unit angle.

Strain Hardening: The phenomenon of increase in stress with increase in strain beyond yielding.

Strength: Resistance to failure by yielding or buckling.

Stress: The internal force per unit area of the original cross-section,

Tensile Stress: The characteristic stress corresponding to rupture in tension specific grade of steel.

Yield Stress: The characteristic stress of the material in tension before the elastic limit of the material is exceeded.

1.2 NOTATIONS

Following symbols and notations shall apply to the provisions of this section.

b Flange width of an I section

b_p Panel-zone width between column flanges

d Depth of an I section

d_p Panel-zone depth between continuity plates

E Modulus of elasticity of steel

f_{ye} Expected yield stress of steel

f_y Design yield stress of steel

KL Effective length of the member

L Length of beam

M_d Design bending moment

M_E Induced bending moment due to earthquake loads

M_j Moment capacity of a joint

M_G Induced bending moment due to gravity loads

M_{pb} Plastic moment of beam member

M_{pbA} Plastic moment at one beam end

M_{pbB} Plastic moment at another beam end

M_{pc} Plastic moment of column member

P_d Design axial force

P_G Induced axial force due to gravity loads

P_E Induced axial force due to earthquake loads

r Radius of gyration

R_d Resistance of connection

R_y Plastic resistance of the connected member based on design yield stress of the material

t Thickness of column web or doubler plate

t_f Thickness of flange

t_w Thickness of web

V_G Shear force due to gravity loads

V_j Shear force at joint

ϑ Design Overstrength factor

γ_{ov} Material Overstrength factor

θ_p Link rotation angle

1.3 UNITS

Unless otherwise noted, this code uses SI units of kilograms, metres, seconds, Pascals and Newtons (kg, m, s, Pa, N).

2 Materials

2.1 Grade of Material

Structural Steels having yield strengths (f_y) of 250 MPa and 345 MPa shall only be permitted to be used for elements resisting the earthquake forces.

2.2 Expected Material Strength

The required strength of a member or a connection shall be determined from the expected yield stress (f_{ye}) of the member or an adjoining member as applicable. The expected yield stress is defined in terms of material over-strength factor (γ_{ov}) as follows:

$$f_{ye} = \gamma_{ov} f_y \quad \text{.....2.1}$$

The expected yield stress can be determined from actual measurements. In absence of any measurements, the material over-strength factor can be taken as 1.25.

2.3 Section Requirements

The structural steel sections resisting the earthquake forces shall be plastic or compact only.

3 Connections

3.1 Bolted and Welded connections

a) The design of the connections shall be designed so as to limit localization of plastic strains, high residual stresses and prevent fabrication defects.

b) Connections fabricated by means of full penetration butt weld is deemed to satisfy the Overstrength criterion.

c) Fillet weld or bolted connections shall be designed by considering the Overstrength and strain hardening of the material as follows:

$$R_d \geq 1.1\gamma_{ov} R_y \quad \text{.....3.1}$$

Where,

R_d = resistance of connection

R_y = plastic resistance of the connected member based on design yield stress of the material

γ_{ov} = Material Overstrength

d) Bolts and welds shall not be designed to share force in a joint or the same force component in a connection.

3.2 Column Splices

a) Column Splices shall be located away from the beam to column flange connection. The splice shall be located in the middle half of the clear height of the column.

b) Splices shall be either bolted or welded or welded to one column and bolted to the other.

c) Splice Plates or Channels used for making web splice shall be placed on both sides of the column web.

d) When partial-joint penetration groove weld is used, the design strength of the joints shall be at least 200% of the required strength.

e) For welded-butt joint splices made with groove welds, weld tabs shall be removed. Steel backing of groove welds need not be removed.

3.3 COLUMN BASES

a) Axial Strength

Column Bases shall be designed for column axial load calculated using the design overstrength factor.

b) Shear Strength

The required shear strength shall be as follows:

(i) For diagonal braces, required shear strength shall be equal to the horizontal component of the brace strength.

(ii) For Columns, lesser of

a) Shear force calculated using design overstrength factor

b) $1.1\Upsilon_{ov}V_p$ 3.3.1

c) Flexural Strength, lesser of

a) Flexural strength calculated using design overstrength factor

b) $1.1\Upsilon_{ov}M_p$ 3.3.2

4 Location of Plastic Hinges

a) In moment resisting frames, the plastic hinges shall be primarily located at the ends of the beam member. It is permitted to locate plastic hinges in columns at the following locations:
    i. at the base of the frame
    ii. at the top of the columns in upper story of multi-story buildings
b) In frames with concentric braces, plastic hinge shall be primarily located in the tension diagonals. In case of V-bracings, plastic hinges are permitted to be located in both tension and compression diagonals.
c) In frames with eccentric braces, plastic hinges shall be located in the links. It shall be ensured that plastic hinges shall be formed in all the links simultaneously.

5 Moment Resisting Frames

5.1 BEAMS

Beams shall have sufficient resistance against lateral and torsional buckling. Abrupt changes in the beam flange area are prohibited in plastic hinge regions.

5.2 COLUMNS

Columns shall be checked for the most unfavorable combination of axial force, shear force and bending moments. The column shall be checked for the following load combinations:

$$P_d = P_G + 1.1\gamma_{ov}\vartheta P_E \quad \text{.....5.2.1}$$

$$V_d = V_G + 1.1\gamma_{OV}\vartheta V_E \quad \text{.....5.2.2}$$

$$M_D = M_G + 1.1\gamma_{OV}\vartheta M_E \quad \text{.....5.2.3}$$

Where,

P_d = Design axial force

P_G = Induced axial force due to gravity loads

P_E = Induced axial force due to earthquake loads

M_d = Design bending moment

M_G = Induced bending moment due to gravity loads

M_E = Induced bending moment due to earthquake loads

γ_{ov} = Material Overstrength factor = 1.25

ϑ = Design Overstrength factor = 2.5

5.3 BEAM TO COLUMN CONNECTIONS

a) All beam to column connections shall be rigid.
b) The connection shall be designed in such a manner to form plastic hinges at beam ends.
c) The connection shall be designed for the resulting moment calculated as per following equation:

$$M_j = 1.1\gamma_{OV} M_{pb} \quad \text{.....5.3.1}$$

Where,

M_{pb} = Plastic moment of beam member

γ_{ov} = 1.25

M_j = Moment capacity of joint

d) The connection shall be designed for a shear force resulting from the following load combination:

$$V_j = V_G + \left(\frac{M_{pbA}+M_{pbB}}{L}\right) \quad \text{.....5.3.2}$$

Where,

M_{pbA} = Plastic moment at one beam end

M_{pbB} = Plastic moment at another beam end

V_G = Shear force due to gravity loads

V_j = Shear force at joint

L = Length of beam

e) In column strong axis connections (beam and column web in the same plane), the panel zone shall be checked for shear buckling. Column web doubler plates or diagonal stiffeners may be used to strengthen the web against shear buckling. The individual thickness of the column webs and doubler plates, shall satisfy the following:

$$t \geq \frac{(d_p+b_p)}{90} \quad \text{.....5.3.3}$$

Where,

t = thickness of column web or doubler plate,

d_p = panel-zone depth between continuity plate, and

b_p = panel-zone width between column flanges

f) Continuity plates (tension stiffener) shall be provided in all strong axis welded connections except in end plate connections.

5.4 COLUMN-BEAM MOMENT CAPACITY RATIO

At all beam to column connections, the following relationship shall be satisfied:

$$\frac{\sum M_{pc}}{\sum M_{pb}} \geq 1.2 \quad \text{.....5.4}$$

Where,

ΣM_{pc} = sum of the moment capacity in the column above and below the beam centerline;

ΣM_{pb} = sum of the moment capacity in the beams at the intersection of the beam and column centerlines

6 Frames with Concentric Braces

6.1 CONCENTRIC BRACES

Concentric braces shall be designed so that the yielding of the tensile diagonals or reaching the post-buckling compressive strength in compression diagonals will take place before failure of connections and before yielding or buckling of the beams or columns. Three types of braces are considered in this standard:

1. Diagonal X-Type Bracing
2. V-Type Bracing
3. Inverted V-Type Bracing

K-type bracing is not permitted to be used for bracing.

6.1.1 Slenderness ratio

The slenderness ratio of the bracing members shall satisfy the following:

a) For diagonal X-Type Bracing

$$4\sqrt{\frac{E}{f_y}} < \frac{KL}{r} \leq 6\sqrt{\frac{E}{f_y}} \quad \text{.....6.1.1}$$

b) For V-Type and Inverted V-Type Bracing

$$\frac{KL}{r} \leq 6\sqrt{\frac{E}{f_y}} \quad \text{.....6.1.2}$$

6.1.2 Placement of braces

The diagonal braces shall be placed in such a way that the structure exhibits similar load deflection response in each story under earthquake load applied in either direction.

6.1.3 Built-up Member braces

a) The spacing of stitches shall be such that the slenderness ratio of individual element between such stiches shall not exceed 0.4 times the governing slenderness ratio of the Built-up brace itself.
b) The spacing of such stiches shall be uniform.
c) The sum of available shear strengths of the stitches shall be equal to or exceed the available tensile strength of each element.
d) Bolted connections shall be avoided within the middle one-fourth of the clear brace length (0.25 times the length, in the middle).

6.1.4 Brace Connection

The brace connection shall satisfy the requirements specified in clause 3.

6.2 BEAMS AND COLUMNS

The beams and columns adjacent to the braces shall be checked for the following load combinations:

$$P_d = P_G + 1.1\gamma_{ov}\vartheta P_E \quad \text{.....6.2.1}$$

$$M_D = M_G + 1.1\gamma_{OV}\vartheta M_E \quad \text{.....6.2.2}$$

Where,

P_d = Design axial force

P_G = Induced axial force due to gravity loads

P_E = Induced axial force due to earthquake loads

M_d = Design bending moment

M_G = Induced bending moment due to gravity loads

M_E = Induced bending moment due to earthquake loads

γ_{ov} = Material Overstrength factor = 1.25

ϑ = Design Overstrength factor = 2.0

7 Frames with Eccentric Braces

Frames with eccentric braces shall be designed in such a manner that the specific "seismic links" are able to dissipate energy by formation of plastic bending and shear mechanisms. Seismic Links may be horizontal or vertical.

7.1 SEISMIC LINKS

Seismic links are classified into 3 categories according to the type of the plastic mechanism developed:

a) Short Links: Short links dissipate energy by yielding essentially in shear.
b) Long Links: Long links dissipate energy by yielding essentially in bending
c) Intermediate Links: Intermediate links dissipate energy by yielding in both bending and shear.

7.1.1 Link Section Limitations

Links shall be of I sections only. Either hot rolled I sections or built-up I sections are permitted to be used.

For I sections, the following equations are used to calculate the design resistances and limits of categories:

$$M_{p,link} = f_y\, b\, t_f (d - t_f) \quad \text{.....7.1.1}$$

$$V_{p,link} = \left(\frac{f_y}{\sqrt{3}}\right) t_w (d - t_f) \quad \text{.....7.1.2}$$

Where,

b = Flange width of the I section

t_f = thickness of the flange

d = depth of the I section

t_w = thickness of the web

If $P_d/P_c \leq 0.15$, the design resistance of the link should satisfy the following relationships at both ends of the link:

$$V_d \leq V_{p,link} \quad \text{.....7.1.3}$$

$M_d \leq M_{p,link}$ 7.1.4

Where, P_d, M_d, V_d are the design action effects, respectively the design axial force, design bending moment and design shear, at both ends of the link.

If $P_d/P_c > 0.15$, the following reduced values of $V_{p,link,r}$ and $M_{p,link,}$ shall be used instead of $V_{p,link}$ and $M_{p,link}$

$$V_{p,link,r} = V_{p,link}\left[1-\left(\frac{P_d}{P_c}\right)^2\right]^{0.5} \quad \text{.....7.1.5}$$

$$M_{p,link,r} = M_{p,link}\left[1-\left(\frac{P_d}{P_c}\right)\right] \quad \text{.....7.1.6}$$

If $P_d/P_c \geq 0.15$, the link length shall not exceed:

$e \leq 1.6\frac{M_{p,link}}{V_{p,link}}$, when R < 0.3 .....7.1.7

or

$e \leq (1.15-0.5R)1.6\frac{M_{p,link}}{V_{p,link}}$, when R≥ 0.3.....7.1.8

Where

$$R = P_d t_w \frac{(d-2t_f)}{(V_d A)} \quad \text{.....7.1.9}$$

in which A is the gross area of the link.

7.1.2 Horizontal Seismic Links

In horizontal seismic links, plastic hinge would form simultaneously at both ends of the links. The links shall be categorized according to the length e:

Short Links:

$$e \leq 1.6\frac{M_{p,link}}{V_{p,link}} \quad \text{.....7.1.10}$$

Long Links:

$$e \geq 3\frac{M_{p,link}}{V_{p,link}} \quad \text{.....7.1.11}$$

Intermediate Links:

$$1.6\frac{M_{p,link}}{V_{p,link}} < e < 3\frac{M_{p,link}}{V_{p,link}} \quad \text{.....7.1.12}$$

7.1.3 Vertical Seismic Links

In vertical seismic links, only one plastic hinge would form at one end of the link. The links shall be categorized according to their length:

Short Links:

$$e \leq 0.8\,(1+\alpha)\frac{M_{p,link}}{V_{p,link}} \quad \text{.....7.1.13}$$

Long Links:

$$e \geq 1.5\,(1+\alpha)\frac{M_{p,link}}{V_{p,link}} \quad \text{.....7.1.14}$$

Intermediate Links:

$$0.8\,(1+\alpha)\frac{M_{p,link}}{V_{p,link}}\ e < 1.5\,(1+\alpha)\frac{M_{p,link}}{V_{p,link}} \quad \text{.....7.1.15}$$

Where,

α = ratio of the smaller bending moment at one end to the greater bending moment at the other end.

7.1.4 Link Rotation Angle

The link rotation angle θ_p between the link and the element outside of the link should be consistent with global deformations. It should not exceed the following values:

a) short links $\theta_p \leq 0.08$ radians
b) long links $\theta_p \leq 0{,}02$ radians
c) Intermediate links θ_p shall be determined by linear interpolation between the above values

7.1.5 Web Stiffeners

Full-depth web stiffeners should be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners shall have a combined width of not less than ($b_f - 2t_w$) and a thickness of not less than $0{,}75t_w$ nor 10 mm, whichever is larger.

Links should be provided with intermediate web stiffeners as follows:

a) Short links should be provided with intermediate web stiffeners spaced at intervals not exceeding ($30t_w - d/5$) for a link rotation angles θ_p of 0.08 radians or more, or ($52t_w - d/5$) for link rotation angles θ_p of 0,02 radians or less. Linear interpolation should be used for values of θ_p between 0.08 and 0.02 radians;

b) Long links should be provided with one intermediate web stiffener placed at a distance of 1.5 times b_f from each end of the link;

c) Intermediate links should be provided with intermediate web stiffeners meeting the requirements of a) and b) above;

d) Intermediate web stiffeners are not required in links of length greater than 5 M_p/V_p;

e) Intermediate web stiffeners should be full depth. For links that are less than 600 mm in depth, stiffeners are required on only one side of the link web. The thickness of one-sided stiffeners should be not less than t_w or 10 mm, whichever is larger, and the width should be not less than $(b_f/2) - t_w$. For links that are 600 mm in depth or greater, similar intermediate stiffeners should be provided on both sides of the web.

Fillet welds connecting a link stiffener to the link web should have a design strength adequate to resist a force of $\gamma_{ov} f_y A_{st}$, where A_{st} is the area of the stiffener. The design strength of the fillet welds fastening the stiffener to the flanges should be adequate to resist a force of $\gamma_{ov} A_{st} f_y/4$.

Lateral supports should be provided at both the top and bottom link flanges at the ends of the link. End lateral supports of the links should have a design axial resistance sufficient to provide lateral support for forces equal to 6% of the expected nominal axial strength of the link flange computed as $f_y\, bt_f$.

7.2 BEAMS, COLUMNS AND DIAGONAL MEMBERS

Members not containing seismic links shall be designed for axial load from the following load combinations:

$$P_D = P_G + 1.1\gamma_{ov}\vartheta P_E \quad \text{.....7.2.1}$$

Where,

P_d = Design axial force

P_G = Induced axial force due to gravity loads

P_E = Induced axial force due to earthquake loads

γ_{ov} = Material Overstrength factor = 1.25

ϑ = Design Overstrength factor = 2.0

In case of horizontal seismic links, columns and diagonal members shall be checked for such load combination. In case of vertical seismic links, columns, beams and diagonal members shall be checked for such load combination.

7.3 CONNECTION TO SEISMIC LINKS

The connection to the seismic links shall be designed using the following rule:

$$R_d \geq 1.1\gamma_{ov}\vartheta R_y \quad \text{.....7.3.1}$$